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with the direction of the PUBLICATIONS COMMITTEE: W. A. SLATER, chairman; DUFF A. ABRAMS, P. H. BATES, S. C. HOLLISTER, A. R. LORD, A. N. TALBOT, BENJAMIN WILK, HARVEY WHIPPLE, Secretary.

Papers: Original papers, furthering the objects of the society, are invited. They should be addressed to the Secretary, and their consideration by the Publications Committee will be facilitated if: (1) a brief but comprehensive outline or summary precedes or at least accompanies the completed paper; (2) manuscript and photographs are available in duplicate or triplicate; (3) original tracings or other drawings, from which copies are readily made, are available.

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AMERICAN CONCRETE INSTITUTE

Founded 1905; incorporated 1906 (National Association of Cement Users); charter amended 1913. (American Concrete Institute.)

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*The five latest, living past presidents are members of the Board.

VOL. 1—NO. 4. FEBRUARY 1930 (*Proceedings Vol. 26*)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

News Letter

The March NEWS LETTER *will carry a running story of the 26th Annual Convention New Orleans February 11, 12, 13.*

THANK YOU

THE EDITOR wishes to pay his respects to those who have made it possible to issue this JOURNAL before the opening of the Institute's 26th Annual Convention.

Before the December JOURNAL got into the mails—late beyond all reasonable excuse—we told Harry Friesema, president of Friesema Brothers Printing Co., Detroit, that we wanted his cooperation in getting out two JOURNALS —January and February—each of them bigger than either November or December, by February 6. In giving his promise Mr. Friesema virtually signed a blank contract, because much of the "copy" was still commanding the overtime efforts of its authors. To these authors, our gratitude. To those Institute members who *receive* much more often than they *give*, this thought: The Institute owes its effectiveness to those in whom some sense of responsibility to a cause, some indefatigable loyalty to the purpose of this Institute, keep the mills grinding a fresh grist of experience. Worthy papers add to their authors' professional standing and it is something worth while to be integral with the growth of knowledge in one's chosen field. It simmers down to this: A few feel "the urge" which is its own reward. Those who haven't it can't understand it. Those who have it know that it makes short days out of long ones and quickens the sense of living. Our hats off to those who have "the urge."

As this editorial postscript is written we can't be sure that the printer will make good on the promised date; the assumption, based on past experience, is that he will not be far behind.

FUTURE COMMITTEE WORK—YOUR IDEAS NEEDED

THE ADVISORY COMMITTEE of the Institute is the clearing house for ideas in reference to committee assignments. It has before it a number of suggestions for new work the Institute might undertake, and it has seemed wise to pass these ideas on to members of the Institute, in the hope that here and there a member will react helpfully to the suggestions being considered. Such reaction may take the form of suggestions to expand or otherwise modify committee assignments or indicate men who might be useful in the studies proposed.

RESEARCH

In Department 100, Research, it has been recommended in the Advisory committee that committees be formed to study the following subjects:

1. High strength concrete by vibration. Exceedingly dry concrete is now being placed on large jobs economically. It is proposed to ask the committee to investigate properties of concrete and the best means of vibrating. A parallel investigation would be in the field of concrete products manufacture where vibration machinery is aiso in use and a subject of lively interest with a view to still wider use.

2. Effect of accidental misplacing of stirrups. Should welded units be required?

3. Absorption of concrete. Study the effect of pore shapes, sizes and distribution; methods of water proofing surfaces of concrete blocks etc. This is particularly for use with units of porous, light-weight aggregate.

4. Chemical reaction with concrete. Studies of the effect on concrete of various degrees of exposure to chemicals commonly met with in commercial practice of one kind or another where concrete is used.

MATERIALS

In Department 200, Materials, it has been suggested that a study be undertaken of variations in the concrete making properties of aggregates; from single sources, from a single neighborhood and from widely separated sources.

2. Spirals for column reinforcement. Use of cold drawn wire vs. hot rods; maintaining true cylindrical shape at all points during handling, etc.

3. Cinders as an aggregate for concrete. Study of probable variations and their elimination; construction and design problems peculiar to the use of cinder as aggregate; the proper selection of cinders for use in concrete masonry units.

4. Grading of aggregates for handling on job. On some current work the coarse aggregate is limited to that retained on a 3-in screen, to secure non-ravelling stock piles. What are the advantages and disadvantages of this method from every standpoint?

5. Burned clay aggregate. A study as a basis for dependable standardization; concrete making properties; variations in these new light weight aggregates.

ENGINEERING DESIGN

In Department 300, Engineering Design, it is proposed that the following studies be made, each in a separate committee:

1. Effect of shrinkage and plastic flow on design considerations. A study of all types of members, both with and without compressive reinforcement.

2. Temperature reinforcement. Proper amount for various conditions; most effective distribution. Relative value of rods and welded mesh.

3. Moment of inertia of reinforced concrete members. Study of effect of reinforcement on stiffness, effect of cracked concrete in tension face of beam; effect of brackets, etc.

4. Effect of settlement of foundations on reinforced concrete structures. Study of actual experience, rate of settlement; effect of plastic coefficient of the concrete, etc.

5. Minimum permissible thickness of slabs and walls. Study of joint considerations of strength, rigidity and fire safety. In this connection someone has asked why a fire wall must be so much thicker than a slab between floors.

6. Basis of design for hurricane exposures. A study of south Atlantic and Gulf Coast problems; should stresses be lowered, as has been done, or should the proper mechanics of such action be used with regular stresses? What is the proper basis of design?

ARCHITECTURAL DESIGN

In Department 400, Architectural Design, the following committee assignments are under consideration:

1. Making of plaster molds and handling on the job, in monolithic construction. Study of best ways to use molds for architectural details, for sculptural decoration on monolithic concrete work.

2. Sound insulating concrete. Study of available aggregates and of methods of placing concrete that may result in low transmission of sound; relation of sound insulation to acoustics.

3. Painting on concrete surfaces. Study of necessary preparation of concrete to receive various types of paint for permanent results.

4. Causes of changes in color of concrete surfaces. Study of effect of form lumber; surface treatment of forms; mixing and placing concrete, aggregates, curing, etc.

5. Use of light weight concrete in building. Study of economic and architectural fitness of cinders and Haydite Blocks and Haydite concrete. Safeguards.

SPECIFICATIONS

In Department 500, Specifications, the following work is proposed:

1. Concreting in the presence of water. Study of special problems of construction and supervision occasioned by water in deep pier walls, foundations close to open water, etc.

2. Specifications for proper winter protection of concrete. Study of minimum, fully effective measures to be written into the contract specifications for buildings. Basis will be available in the work of committee 604.

3. Purchase specifications for metal supports for reinforcement. Study of minimum strength requirements, springiness, stiffness, kind of metal to be used, etc. to secure results.

4. Specifications for architectural concrete surfaces.

FIELD CONSTRUCTION

In Department 600, Field Construction, the following studies are proposed:

1. High early strength concrete in the field. Study of the cost of various methods available to produce such concrete; saving in patching. Saving in early form work removal and re-use.

2. Sliding form work. Study of range of application, speed, quality of resulting concrete, practical operations, cost of work under different conditions.

3. Protecting concreting operations in stormy weather. Study of effect of rain on fresh concrete just after or during finishing. Methods of patching craters and sandy surfaces.

4. Job correction for moisture in aggregates. Study of common variation in amount of moisture, simple quick tests for checking, tables or diagrams to apply correction.

5. Design of form work to relieve shrinkage stresses at openings. Study of yielding form work design, early removal of wedges to allow concrete to shrink freely.

6. Standards for workability on the job. Study of design and use of device to determine effect on workability of changes in mix, water, admixtures, etc.

7. Use of bulk cement. Methods and equipment for handling and storing; economic limitations.

8. Storage of cement. A restatement of the answer to the question "How long can one safely store cement"; practical rules for its storage in the field. There are a large number of jobs distant from cement mills where this is a very real and important problem and the number of inquiries on this point indicate a demand for such information.

9. Methods of placing concrete in the form. The methods used in placing have a great deal to do with the character and surface of the exposed concrete and its frequent blemishes; the cost of pointing and finishing. The methods of placing greatly influence its subsequent durability. The whole question of handling and placing might well be studied by a committee of the Institute. As a corrollary to this there is the special problem of handling, depositing and consolidating the dry concrete now sometimes required in heavy mass work.

SHOP MANUFACTURE

In Department 700, Shop Manufacture, the following suggestions

1. A study of broken ashlar masonry units. Sizes of blocks. various means of waterproofing surfaces without losing texture and jointing; details of various types of floors, windows, etc.

2. Analytical study of operating problems in plants for the manufacture of standard concrete masonry units.

3. Most important perhaps in the concrete masonry unit field is the work to some extent anticipated in an item on the program of the 26th annual convention, the report of committee 708 involving a review of the Institute's five year old recommended practice for the manufacture of standard concrete masonry units. This review discloses: that characteristics of masonry units now of major importance should be carefully considered in the tentative recommended practice of the Institute in this field; that there needs to be a complete re-analysis of outstanding considerations in the manufacture of concrete masonry units.

USE REQUIREMENTS

Department 800, Use Requirements:

1. Efficiency of expansion joints. Study of joints used and of cost and effectiveness over long period of time of each type.

2. Roof insulation. Study of material used and of cost and effectiveness. Stability of parapet walls, etc.

3. Use of precast concrete units on railroad work. A review of the variety of concrete objects now used at stations and on right of way.

The foregoing are being considered by the Advisory committee in reference to possible work for the near future.

If members of the Institute have suggestions in reference to any of these subjects, or in reference to any subject not here listed, will they please communicate with the Secretary of the Institute so that the matter may be placed before the Advisory committee.

Honor Roll to January 22, 1930—9 Days to Go

Members of the Institute active in membership work are listed here, with a number set opposite each name representing the new members sponsored since Feb. 1, 1929.

John G. Ahlers	**15**
W. F. Way	**11**
I. E. Burks	**10**
H. F. Gonnerman	**10**
H. B. Emerson	**6**
George H. Fenkell	**6**
M. M. Upson	**6**
E. Y. Bragger	**4**
F. R. McMillan	**4**
Duff A. Abrams	**3**
W. D. M. Allan	**3**
L. E. Andrews	**3**
G. J. Eyrick, Jr.	**3**
H. J. Gilkey	**3**
A. R. Lord	**3**
Alan Moncrieff (India)	**3**
Hale Sutherland	**3**
Miguel Villa (Cuba)	**3**

Almost on the Roll:

F. C. Brookhouse	2
J. Campbell & Sons Ltd.	2
B. E. Clark	2
A. K. Dandekar	2
E. W. Dienhart	2
G. L. Freeman	2
T. Campbell Gray	2
W. E. Hart	2
J. C. Hawkins	2
W. S. Loucheim	2
B. A. Rich	2
F. E. Richart	2
C. J. Seibert	2
John Tucker, Jr.	2

Mr. Burk's Paper and Others "Off the Job"

The editor is under no inhibitions which deny him the right to enthuse. Mr. Burk's paper, "Concreting Methods at Chute a Caron Dam" has just gone to the printers; the monotypers are keyboarding it for other pages of this Journal. It will reach you as soon as do these few lines and be read more promptly. You may judge for yourself how well Mr. Burks has done—by the dam at Chute a Caron, by the record he has given you of the methods used there. So why should the editor record *his* particular enthusiasm? Because there should be more papers "hot off the job." More like Mr. Burks should be writing papers. We don't know what ideas are good for until we get them out on the job. Ideas have to be highly non-abrading to stand a rattler test of hard-driven job practice and temperatures of 50 below. The research people come nearer to bearing their share of the burden of progress than do men on the job. Their meticulous, non-partisan searchings for truth in mazes of graphs and endless tabulations are at the foundation of what Mr. Burks and his kind are accomplishing.

Let the men on the job more often discharge their debts with interest, in such reports from the frontiers of practice as Mr. Burks has sent us from the Saguenay. Let it be hinted broadly to any Institute past-president who considers founding a new Award that there should be a special recognition, in a class all their own, of papers contributed to this Journal, which effectively put *the job* on paper. How-it-was-done information is the rarest information. The translation of theory into practice,

of laboratory to job, of specification to workmanship—these are the true tests of knowledge. The essential elements of such tests are the most difficult to record. Too often the craftsman is inarticulate and the essence of his craft escapes a coarse mesh of words.

It was A. E. Lindau, a seasoned builder of Institute programs, returning from Chute a Caron, who suggested to the editor, "Get a paper from Burks."

A Uniform Method for Testing Portland Cement

BY MAXIMILIAN TOCH*

Up to a short time ago, practically the only test portland cement was put through was to take one part of cement, three parts of Ottawa sand or other sand of the same type, make a briquette of the well known dumb-bell shape and test it for tensile strength at 7, 14 and 28 days.

I am sure that we all realize that this gives us no unit of measure, and while the test is still conducted, tensile strength plays so little part in concrete that even a poor cement, or one that has been overcharged with water in a properly reinforced bed, has ample tensile strength to give it a factor of safety far beyond its ultimate need.

What is much more important is density and compression, for density can be translated into the amount of water pressure that it will stand, and compression is an indicator of the amount of water that it should hold; but we are by no means agreed as to the methods that should be employed for the uniform testing of portland cement. I feel assured that once this idea is started we shall hear from the

*Toch Brothers Inc., New York City

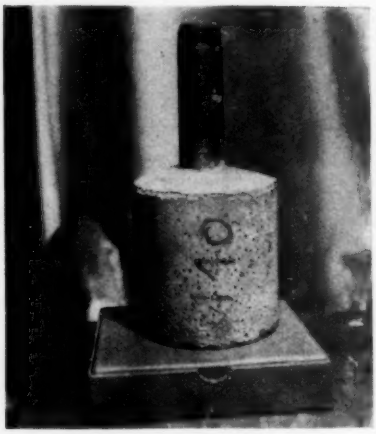


Fig. 1—3 x 3 in. Cylinder for Determining the Pressure that One-Inch Concrete Will Stand.

Europeans. In a reasonably short time I am certain that we can arrive at methods that will give us concrete results.

I do not propose that my methods should be adopted. I am merely sending out this message so that others will communicate. We shall arrive at a system comprehensive to all. The methods I am outlining have been used in our laboratory for a number of years, where we have found it of great importance in determining the value of integral waterproofing agents, accelerators and densifiers.

My first proposal is not to abandon the tensile strength test, but to eliminate as much as possible the human factor in making tests. We all know that we must take three (3) briquettes and take the average, because the human factor enters so much into the making of these briquettes that the results are always discordant. This should not be.

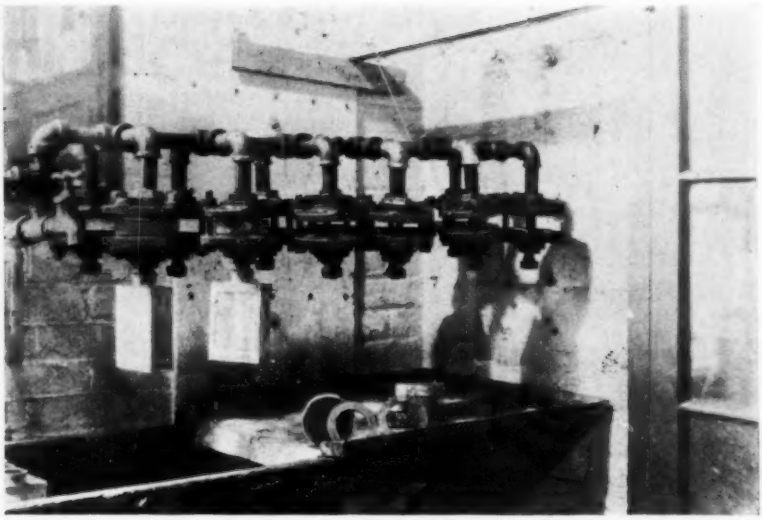


FIG. 2—DISC MACHINE ATTACHED TO CITY WATER HYDRANT—60 LBS. PRESSURE. WATER PASSING CONCRETE IN SPECIFIED TIME IS MEASURED.

My proposals are as follows:

A weighted amount of cement or a measured amount of cement and a measured amount of sand should be placed in a mixer with a definite quantity of water added and a mechanical rotation resorted to for five minutes. The resultant mass should be poured into the molds and flattened out with a palette knife.

It really should make no difference what kind of sand is used. If the same type but not the same kind of sand is used in each laboratory, the results in the one laboratory cannot vary very widely from the results obtained in another laboratory.

The making of the cubes or cylinders for the compressive tests should likewise be done mechanically and to this end a uniform mixer should be adopted. The number of revolutions should be the same in all laboratories; the percentage of water should be the same; the volume of water and sand should be the same then fairly concordant results will be obtained. The same method should be employed for the making of discs in determining density and water pressure.

In addition to that, cylinders 3 x 3 in. as shown in the Fig. 1 can be attached by means of a piece of pipe to city water hydrants, (Fig. 2) the pressure of which can be easily determined by a gauge or by communicating with the Water Department and the amount of water which trickles through within a certain time can be accurately noted. In addition to that, one-half a gram of phenothalein can be inserted in the tube before the water is allowed to percolate and the resultant moisture or water

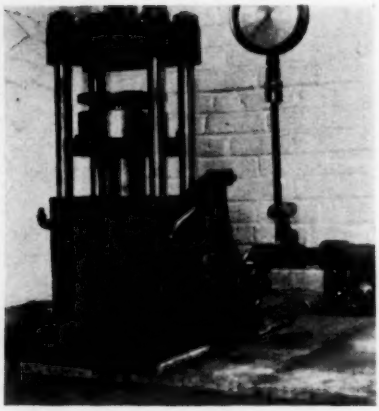


FIG. 3—APPARATUS FOR DETERMINING COMPRESSIVE STRENGTH.

that comes through will be indicated in a purplish red color.

This is merely an outline of what I should like to see done at a conference of the Institute, in order that a method may be devised which is accurate and intelligent.

THE COLUMN TEST FUND

AS THIS issue of the JOURNAL goes to press, cash contributions to the Reinforced Concrete Column Test Fund are $10,352.50—$5,352.50 raised as a result of funding efforts by Henry C. Turner—of this latter amount, $3,537.50 being credited to the New York City Committee with John G. Ahlers as local chairman. Much more is needed.

CONTRIBUTIONS RECEIVED FOR REINFORCED CONCRETE COLUMN TEST FUND

New York City Local Committee

Received since January 10

Ford, Bacon & Davis	**$100.00**
Francisco & Jacobus	**50.00**
Mitchell, Inc., James, Jersey City, N. J.	**150.00**
Purdy & Henderson	**50.00**
Raymond Concrete Pile Co.	**400.00**
Starrett Bros.	**100.00**
United Engineers & Constructors, Inc.	**100.00**
	$950.00

Previously acknowledged

Barney-Ahlers Constr. Corp.	$250.00
Ferguson Co., John W., Paterson, N. J.	250.00
Foundation Co., The	100.00
Industrial Engineering Co.	100.00
Ley & Co., Fred T.	100.00
Lockwood Green Engineers, Inc.	37.50
MacArthur Concrete Pile Corp.	100.00
McCormack Sand Co., Wm. J.	100.00
McGraw & Co., F. H.	100.00
Nassau Sand & Gravel Co.	100.00
New York Trap Rock Corp.	100.00
O'Brien Brothers, Inc.	100.00
Ransome Concrete Machinery Co., Dunellen, N. J.	100.00
Rosoff Sand & Gravel Corp.	100.00
Snare Corp., Frederick	100.00
Spencer, White & Prentis, Inc.	100.00
Steers, Inc., Henry	100.00
Stewart & Co., Inc., James	250.00
Turner Construction Co.	100.00
White Construction Co.	200.00
Goodwin - Gallagher Sand & Gravel Corp.	100.00
	$2,587.50
New York City Local Committee total to Jan. 28, 1930	**$3,537.50**

Chicago

Universal Atlas Cement Co.	**$250.00**

Previously Acknowledged

Turner Construction Co.	$100.00
Chicago total to Jan. 28, 1930	**$350.00**

Philadelphia

Riehle Bros. Testing Laboratories	**$100.00**

Previously Acknowledged

Turner Construction Co.	$100.00
Philadelphia total to Jan. 28, 1930	**$200.00**

Toronto

Alfred Rogers, Ltd.	**$100.00**

CINCINNATI LOCAL COMMITTEE

Previously Acknowledged

Avril Tru-Batch Concrete, Inc.	$25.00
American Aggregates Corp.	25.00
Cincinnati Builders Supply	15.00
Dugan, J. N.	10.00
Euphrat & Hanly	25.00
Ferro Concrete Const. Co.	100.00
Fisher-DeVore Const. Co.	50.00
Hall Co., T. J.	20.00
Hannaford & Sons, Samuel	25.00
Hodges Const. Co.	50.00
Miller & Son, Wm.	5.00
Moores-Coney Corp.	15.00
Nutting Co., H. C.	25.00
Penker Construction Co.	25.00
Van Camp Stone Co.	25.00
	$440.00

MONTREAL

Anglin-Norcross, Ltd.	25.00
Canada Cement Co.	200.00
Peck Rolling Mills	100.00
Steel Company of Canada, Ltd.	100.00
	$425.00

WAUKESHA, WIS.

Butler Bin Co.	150.00

SAN FRANCISCO

Transit Mixers, Inc.	50.00

BOSTON

Turner Construction Co.	100.00

SPECIAL CONTRIBUTIONS

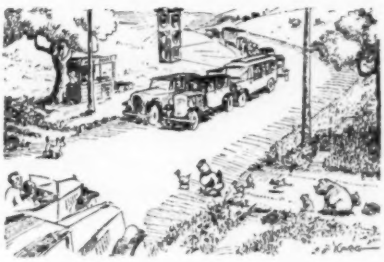
Concrete Reinforcing Steel Institute	$2,500.00
Rail Steel Bar Association	$2,500.00
Total Cash Received to Jan 28, 1930	**$10,352.50**

IN CONSTANT REPAIR

(A Conversation with Dr. Sam'l Johnson)
Dr. Johnson: A man, Sir, should keep his Friendship in a state of Constant Repair.
The Author: And no less, Sir, his Information.

2—TRAFFIC CONTROL

BY ARTHUR R. LORD



RURAL HIGHWAY TRAFFIC CONTROL AS SEEN BY THE NEWARK EVENING NEWS CARTOONIST

THE cartoon reproduced above kept company in the mail with a discussion of the construction specifications for ordinary concrete work on buildings that appeared in the November JOURNAL. There was no stated connection between the two. The discussion, however, was largely an objection to the specifications on the basis that they grossly infringed the contractor's liberty—his right of self determination—and would, in the unlikely event of their general adoption by Architects and Engineers, reduce the market for a certain type of equipment.

I believe that I can see the politely suppressed connection. The important traffic, impatiently held back from its creative rush, represents the contractors, the workers, the producers and distributors of materials and machinery used on concrete work for ordinary buildings. The pert puppy, the

motherly hen with her baby chicks, the contemplative and cautious pig represent the writers of such specifications. Particularly deadly is the "hot dog" stand which very evidently makes no appeal to these bustling builders and sellers. There is nothing worth while, nothing but disgusting hot dogs, to be had at these cross roads—why are we stopped here? Is this a fair picture of the situation? At least it is an interesting one for us to consider for a moment.

Even the writers of specifications are not agreed among themselves as to what to include, how far to bind the contractor. It would be an ideal situation could we stop with the mere statement of the essential condition to be fulfilled. "This Contractor shall deliver a finished structure composed in all its parts of homogeneous, watertight, permanent concrete having an ultimate crushing strength at 28 days of not less than 2000 lb. per sq. in." Why complicate such a simple affair? If a contractor wants to wrestle with widely varying aggregates, with concrete now sloppy, now stiff, with collapsing bar supports and numerous other common difficulties, it is his money that he wastes. Why should any engineer wish to assume the responsibility for *method*, when he can specify the *result* and leave the contractor to secure it as best he may?

To be specific, my friend Lambie, of Cleveland, sees no reason for specifying a batch mixer at all or for requiring a mixing time of one minute. His apparatus mixes concrete much better in a few seconds and produces it in a steady flow. My friend Talbot (Kenneth, not the Doctor) wants the water to be partly introduced during the first few seconds of the mixing time (and proves its benefits) and also sees no reason for wheeling concrete, instead of chuting it directly into the floor forms. My friends Bruce and Healy want to leave the matter of use of admixtures entirely to the contractor, who appreciates "workability" properly, and does not confuse it with "flow" or "slump" like my other friends Abrams and McMillan. I could extend the list at some length, but this serves sufficiently to illustrate the two points of view. Which should prevail?

I personally *wish* that a specification could pass the responsibility over to the contractor in a single, generous, trusting gesture—but fear that such a course would cause me more trouble than the present one. Now I seem to get good concrete work and have only bad words to contend with—then I fear I would get good words and have bad concrete to contend with. Shall we say that contractors are "put out" by our lack of faith in them—that they leave us to enforce good concreting practices solely because we insist on intruding on their proper field? And that all would be "rosy" would we engineers but mind our own business, which is to set up the necessary quality and amount of concrete and to show on plans the necessary amount and distribution of the reinforcement.

Specifications writers, even so provincial and suspicious as I, have taken some steps along the road of faith. We leave the design of the mixer to others. We leave the design and erection of the formwork to the contractor, even though we limit his freedom (but not his complete responsibility) in removing it. How much farther should we go? And how quickly should we progress? Should we drop all the impertinent bars at one move or gradu-

ally reduce the barrier, bar by bar? Are contractors really desirous of taking over the full responsibility, as the spokesmen so far heard (most of whom are not contractors) indicate?

I have had a few experiences in which the present type of specification proved most saving to the owner. These are cases in which the Contractor ignored the specified provisions to the undoing of the concrete. In such cases it is vital that the specification be *worthy* of a good concrete. Would a simple requirement of quality be equally effective protection? Possibly it would be even more effective, and certainly it would be so if the present specified provisions are debatable.

This whole question of traffic control in concreting operations—and the control of materials, equipment, etc. in general—is an important one. It interests me even more strongly as Chairman of the Advisory Committee, than as a practicing engineer. The influence of Institute standards is far reaching. They set an example even when not used by complete reference. I invite the discussion of this point, particularly by contractors, to the end that my knowledge and instruments may be kept in a state of constant repair.

Papers and Reports—In this Issue

Proceedings American Concrete Institute

Vol. 26 Pages 315-512

CONCRETING METHODS AT THE CHUTE A CARON DAM 315
By I. E. BURKS

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Discussion of the contributions in the following 198 pages is scheduled for publication in the JOURNAL *for June 1930 and should be available not later than May 1.*

Concreting Methods at the Chute a Caron Dam

BY I. E. BURKS*

This paper tells of prospecting for materials at 30° below under 4 ft. of snow—how those materials were sampled, chosen, transported, heated, proportioned; how concrete was mixed, conveyed, deposited, protected. Recent visitors returning from the dam site mentioned two things of special interest: (1) The use of large aggregate with a big gap in sizes—a device to prevent raveling in stock piles and bins, harshness and segregation after mixing; (2) the stiff mix placed with vibrators. Mr. Burks explains.—EDITOR.

THE CHUTE A CARON DAM is in Chicoutimi county in the south-central part of the Province of Quebec, Canada. It is on the Saguenay river about 80 miles above its mouth at the St. Lawrence, and 25 miles below Lake St. John. The district is served by the Canadian National Railway which extends as far as the town of Chicoutimi. From June to November the Saguenay is navigable from the St. Lawrence to Chicoutimi. The flow of the river at the dam site ranges from about 20,000 c. f. s. in March and April to as high as 350,000 c. f. s. in May and June. The summer season usually begins late in May and the first freeze may be expected any time after September 1st. The first snow falls about the middle of October, and the atmospheric temperature drops to as low as 40 and 50 degrees below zero in the winter months. In scheduling concrete work it is the practice to have winter protection equipment ready for use early in September.

DESCRIPTION OF PROJECT

Chute a Caron Dam and Power House is being built by the Aluminum Company of America as the first phase of what is

*Concrete Technician, Aluminum Company of America.

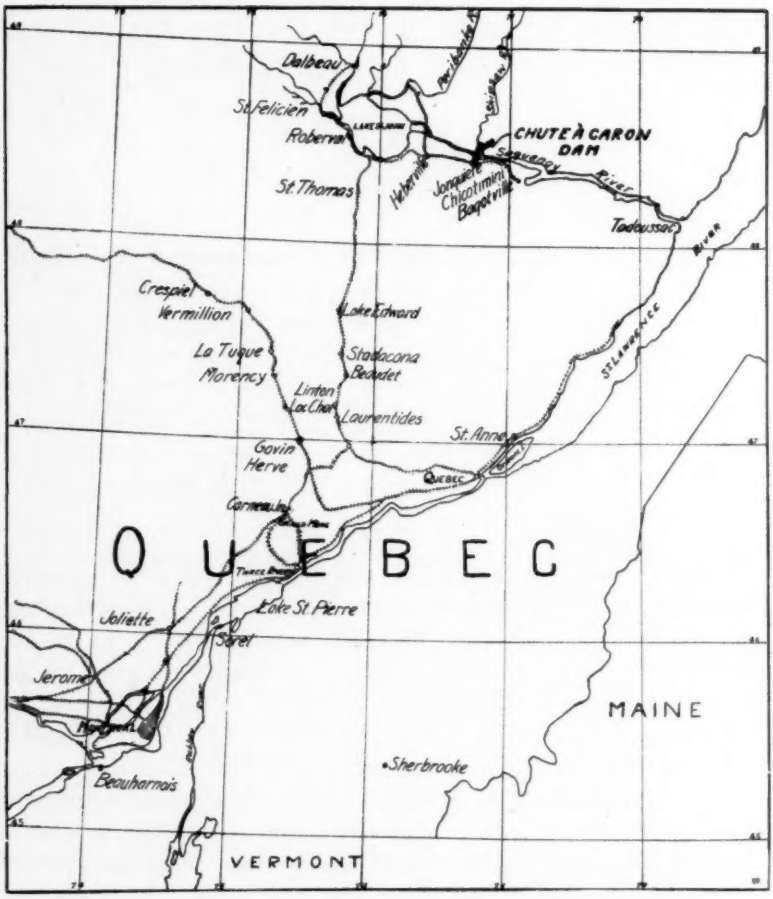


FIG. 1—THE LOCATION OF CHUTE A CARON DAM

known as the Shipshaw development. The dam is the gravity section type and has a total crest length of 3015 ft. The maximum height from river bed is 200 ft. and the greatest width at the base is about 175 ft. In the power house, which is approximately 55 x 300 ft., there will be installed four units of 60,000 h. p. each at 151 ft. head and 120 r. p. m. The estimated quantity of concrete is about 460,000 cu. yds. As this paper is being written,

about 300,000 cu. yds. have been placed. Construction was started late in 1927 and the first concrete placed in June 1928. The project is scheduled for completion in September 1931.

PERSONNEL

The work is being done under the direction of James W. Rickey and James P. Growdon, chief and assistant hydraulic engineers respectively. Ross White is superintendent on the job and J. K. Black is assistant superintendent. The mixing, placing, testing, and general control of the quality of the concrete work is in the hands of a concrete technician who is responsible to the chief hydraulic engineer.

INTRODUCTION

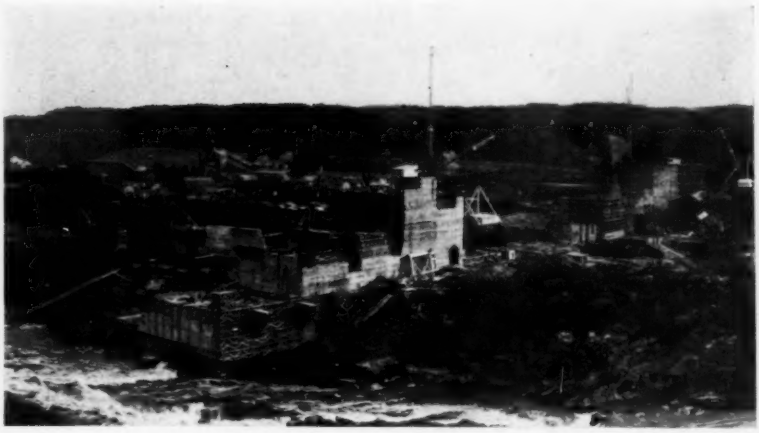
Recognizing the value and necessity for the very highest quality of concrete in its hydraulic structures, and aided and

FIG. 2—THE SAGUENAY RIVER AT FLOOD STAGE

inspired by the research work and teachings of various engineers and investigators, The Aluminum Company of America is making a very determined effort to mix and place concrete in accordance with modern accepted practice. Probably the first step toward scientifically placed concrete was taken in 1917 on the Cheoah Dam. On this structure the old void theory was used

FIG. 3—PANORAMIC VIEW SHOWING PROGRESS

as a basis for mixture design. Later, in 1926, construction on the Santeetlah Dam was started and an effort made to follow the fineness modulus theory of proportioning. During the first few weeks the mixing plant was in operation it was quite evident that any method based on absolute uniformity of aggregate gradation could hardly be successful on work of this magnitude. Since the water-cement ratio law was recognized as a sound basis for mixture design, it was thought that a satisfactory and safe method of proportioning would be to fix the quantity of water per bag of cement in accordance with Abram's curves, and arbitrarily adjust the aggregate quantities to produce the desired workability. This method gave excellent results, was simple in operation, and seemed to be in line with current practice. It has been continued in substantially its same form up to the present time. Once the water-cement law was put into practice, attention was automatically directed toward the effect of aggregate gradation on workability and concrete yield. Due to segregation in the storage bins, batches of coarse aggregate containing an excess of certain sizes would yield a relatively small amount of harsh concrete while other batches containing certain other sizes would yield more than the calculated quantity of a smooth, easy working



OF CONSTRUCTION, AUGUST, 1929

mixture. A number of experiments were made which resulted in the rather unusual grading of the coarse aggregate used on the Chute a Caron job. The matter of gradation will be discussed more in detail elsewhere in this paper.

Having in mind the severe climatic influences which structures in northern Quebec must withstand, a full knowledge of the harmful effect of percolating waters, and with an eye on other structures of similar design which are showing signs of disintegration long before it should be expected, those directly concerned with the placing of concrete in the Chute a Caron Dam set out to build a structure which would stand up indefinitely under these most severe conditions, and which would prove the value of painstaking inspection and careful adherence to modern theories of mixture design and placing methods.

SPECIFICATIONS FOR CONCRETE

Since the construction work was to be carried on by Aluminum Company forces, and the concrete placed under the direct supervision of a Concrete Technician, it was not considered necessary to prepare in advance the usual detailed specifications for the concrete work. The designing office simply specified three

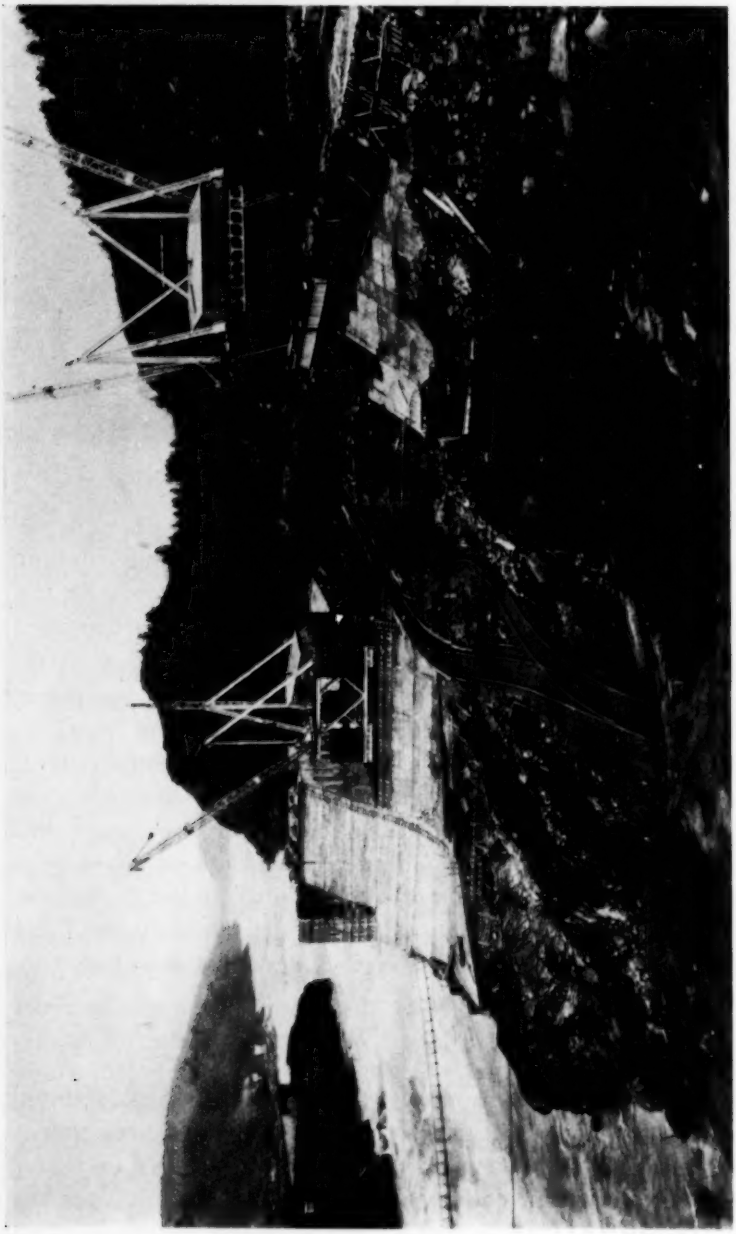


FIG. 4—PROGRESS OF CONSTRUCTION ON THE NORTH SHORE, NOVEMBER, 1928

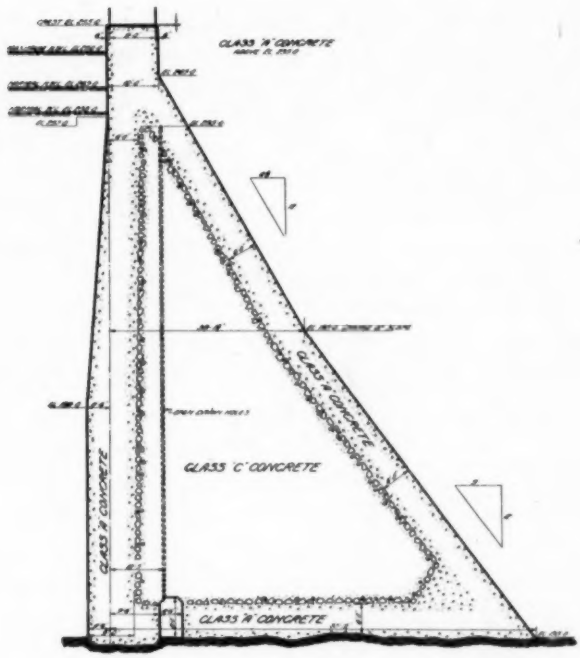


FIG. 5—SHOWING POSITION OF CLASS A AND C CONCRETE IN BULKHEAD SECTION

principal classes of concrete. The matter of quality of aggregates, mixture design, workability placing methods, curing, and winter protection, being left to the judgment of the Concrete Technician. The three classes of concrete specified are as follows:

Class A Concrete—Compressive strength 3000 lb. per sq. in. at 28 days. Maximum size of coarse aggregate 6 in.

Class B Concrete—Compressive strength 3000 lb. per sq. in. at 28 days. Maximum size of coarse aggregate 3 in.

Class C Concrete—Compressive strength 1500 lb. per sq. in. at 28 days. Maximum size of coarse aggregate 6 in.

The strengths designated are based on 6-in x 12-in. laboratory test cylinders modified, as will be explained later, to compensate for the unusual size of the coarse aggregate. The compressive strengths of 3000 lbs. and 1500 lbs. for class A and class C concrete respectively, were specified more for the reason that these

strengths assured to a certain degree the waterproof and weather-resisting qualities desired for the classes indicated—rather than that these strengths were actually called for in the design of the structure. The positions of class A and class C concrete is shown in Figs. 5 and 6. Class B (3-in. coarse aggregate) concrete is intended for use in parts of the dam and power house where the amount and spacing of reinforcing steel and the size of the members precludes the use of the 6-in. coarse aggregate used in class A concrete. From the accompanying sketches it will be noted that class A concrete forms the outer coating or veneer of the structure. Exceptional care is used in proportioning and placing concrete of this class in order to have it as dense and

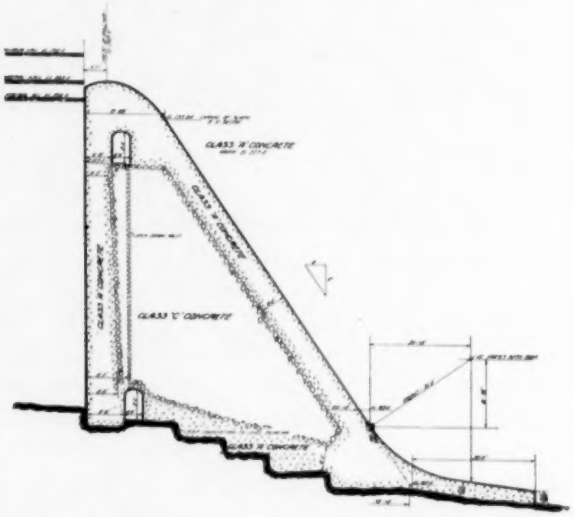


FIG. 6—SHOWING POSITION OF CLASS A AND C CONCRETE IN FREE SPILLWAY SECTION

water-tight as possible. Class C concrete forms the interior mass or core of the dam where the principal requirement is a weight of 150-lbs. per cu. ft.

SELECTION OF AGGREGATES

The original test borings and early excavation work on the project gave evidence that a sufficient quantity of stone of satis-

factory quality would be taken from the various excavations to supply the coarse aggregate for concrete. The geological name of the stone is Anorthosite, described by Prisson and Knopf ("Rocks and Rock Minerals") as a rock composed wholly, or nearly so, of a plagioclase feldspar, usually the variety labradorite. It is noted as being the most abundant representative of the group of igneous rocks that are composed essentially of one mineral. It is found in Canada in separate areas from the west coast of Newfoundland and the east coast of Labrador down through Quebec into eastern Ontario. One of these areas, drained by the Saguenay river, covers nearly 6000 square miles. The particular type of Anorthosite taken from the dam excavations is dark grey and very hard. It produces rather heavy wear on the crushers, screens and conveying equipment but breaks into well shaped fragments for concrete aggregate.

While preliminary studies of the dam site were being made several deposits of satisfactory concrete sand were noted but the extent of these deposits was not developed at the time. About four months before concreting was to begin, exploration parties were organized to put down test pits and procure test samples. The standard by which the sand was to be selected was; (a) it should be structurally sound; (b) free from a harmful amount of organic matter; (c) should not contain more than 3 per cent (by weight) of silt or clay; (d) should have a fineness modulus between 2.5 and 3.3. The first test pits developed that the areas noted in the original survey either did not include a sufficient quantity or the sand did not meet specifications for quality. It was then necessary to extend the work of the exploration parties to investigate other possible sources of supply. The test pit crew, which consisted of from 35 to 50 men, was divided into small groups or reconnaissance parties—each party being directed by a concrete inspector. Well does the writer remember the difficulties encountered in this task, and the feeling of hopelessness that seemed to prevail in the minds of the "prospectors." The ground was covered with from three to five feet of snow which denied the men the aid of surface indications, all cruising had to be done on snowshoes, the thermometer often got down to 30 and 40 below zero and the wind would blow snow into the test pits almost as fast as it could be shoveled out. All test samples had to be carried

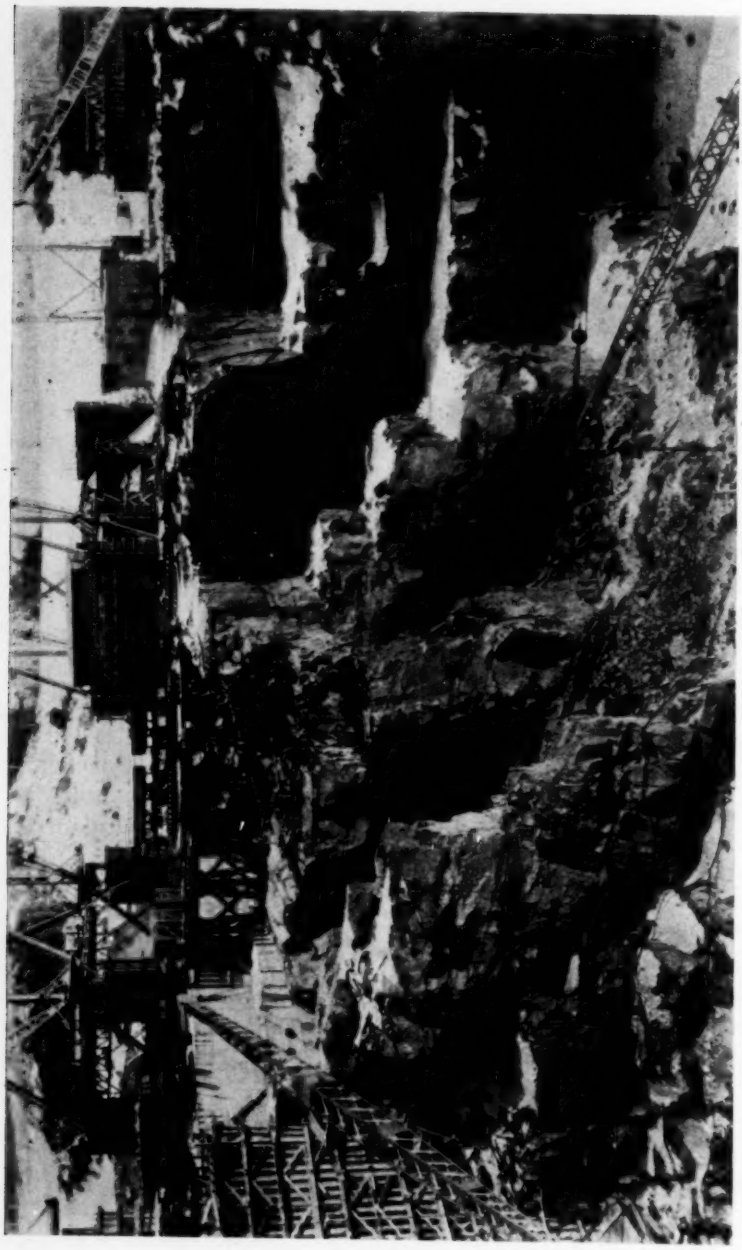


FIG. 7—ROCK EXCAVATION IN POWER HOUSE AREA

two miles or more to the laboratory by dog-team or sled. The procedure in locating test pits was substantially as follows: One of the reconnaissance parties, with the aid of topographical maps and by scratching the surface of the ground here and there, would report a promising area to the gang foreman. Three or four small groups of men would then be detailed to dig as many test pits. The pits would be started 6 ft. long and 4 ft. wide and carried to whatever depth it seemed advisable—some were dug to a depth

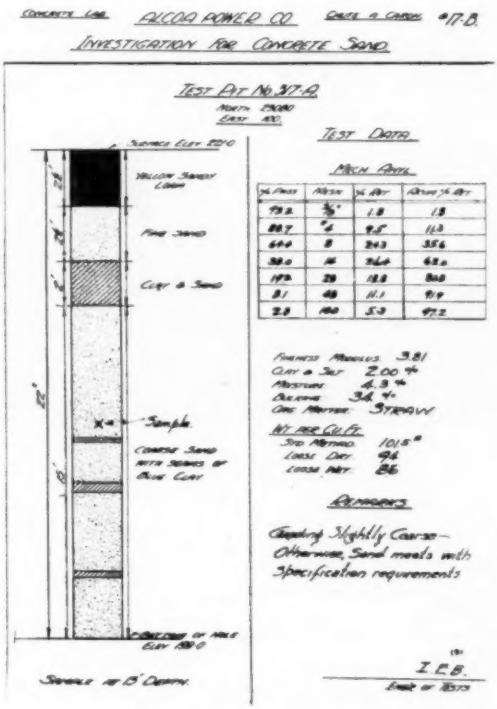


FIG. 8—TYPICAL REPORT ON TEST PIT

of only 3 or 4 ft. and then abandoned while others were made 20 or 30 ft. deep. In one instance a pit was carried to 80 ft. Whenever it was thought advisable to go deeper than about 12 ft., the hole was cribbed and sheeted. The material in each hole was

recorded and sampled—regardless of its quality. The location of each test pit was plotted on the topographical maps and a separate report sheet made out for each hole. Fig. 9 shows a sample of the report form used.

It should be noted that the land over which these investigations were made was originally under the Labrador Ice Sheet and that the sand and gravel deposits are of glacial origin. It was often quite confusing to attempt to follow out the peculiar stratification of the different materials showing up in the test pits. Often a good grade of sand would appear in basin-like pockets, and again in extended beds or strata of varying widths and thicknesses and invariably the sand beds would be streaked with seams, usually 2 or 3 in. thick, of a very dense clay. After over three months of

FIG. 9—THE PEDNAUD PIT

exploration work, three areas of satisfactory sand were located, estimated to meet the demands of the project. These areas are known locally as Pednaud Pit, Price Pit and Carten's Point.

The Pednaud Pit was the nearest (1-mi.) to the mixing plant and required less stripping and track work. It was opened in April 1928 and was used throughout the 1928 season. Approximately 60,000 cubic yards were taken from this area. The grading of the Pednaud sand gave a fineness modulus of from 1.86 to 2.83 which was somewhat finer than was actually desired but it contained very little organic matter and was comparatively clean. The clay and silt content averaged less than 3 per cent.

The Price pit was opened late in 1928 but was not used until May 1929. This area is about 4 miles from the mixing plant.

The pit supplied about 150,000 cubic yards of material, netting approximately 100,000 cubic yards of screened sand—or just about enough to meet the requirement for the 1929 season. The Price sand gave a fineness modulus of from 2.43 to 3.64, somewhat coarser than Pednaud sand. The clay and silt would average slightly above 2 per cent, with occasional pockets showing as high as 4 per cent. Certain sections near the top surface of the pit when examined for organic matter showed a medium brown color under the sodium-hydroxide test. Such sections were rejected.

The Carten's Point area has been reserved for the 1930 concreting season. It is about 5 miles from the mixing plant and its development involves the construction of a 120 ft. bridge across the Shipshaw river and an extension of 0.75 miles to the existing railroad system. The sand from this pit will show about the same grading as the Price Pit, but with less clay and silt and a lower percentage of gravel.

SAND SCREENING PLANT

The sand taken from the Pednaud Pit in the 1928 season contained occasional large boulders but practically no gravel, consequently it was considered unnecessary to pass it through a

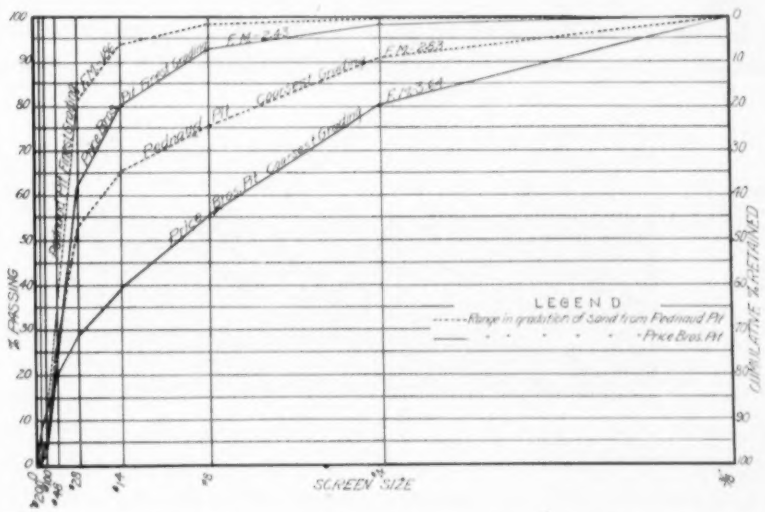


FIG. 10—SHOWING GRADATION OF SAND TAKEN FROM PEDNAUD PIT AND PRICE PIT

screening plant. The boulders were removed by hand as the sand was being loaded on cars.

The pit-run material from the Price Pit, from which sand was obtained for the 1929 season, was not as uniform as the Pednaud sand and contained from 10 per cent to as high as 50 per cent

FIG. 11—THE PRICE PIT

gravel. A screening plant was necessary to separate this material. The plant consists of two sets of bar grizzlies which remove all boulders and cobbles larger than 2½ in. and a vibrating screen having ⅝-in. openings separates the remaining material into finished sand and waste gravel. In Fig. 12 the receiving hopper

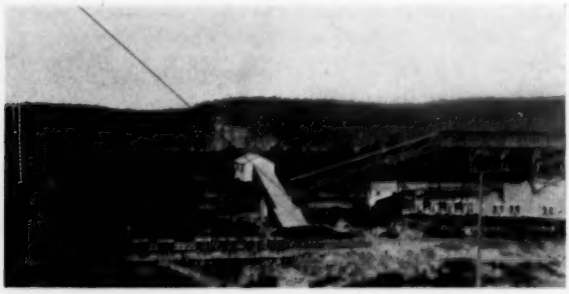


FIG. 12—SAND SCREENING PLANT

and conveyor belt to the screen house are shown in the center, the conveyor belt and storage pile at the left, and the conveyor for the waste gravel is seen at the right. Sand is reclaimed from

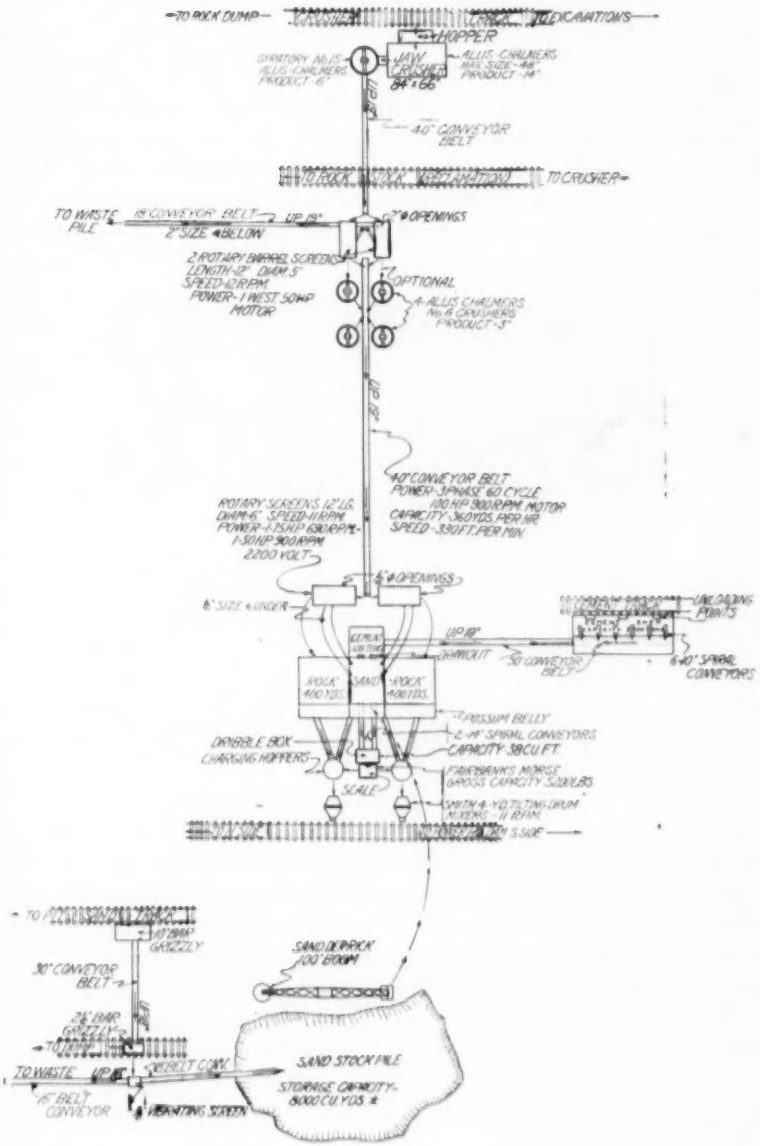


FIG. 13—FLOW SHEET OF PLANT

the storage pile and placed in a bin above the mixers, by a guy-derrick using a 2-yd. clam-shell bucket.

CRUSHING PLANT

The plant in which the coarse aggregate is manufactured consists of a series of crushers, screens, and conveying equipment as shown in Fig. 13. The primary breaker is an 84-in. x 66-in. jaw crusher. This machine will take a 48-in. rock and turn out a 14-in. product. The secondary crusher is a No. 15-N Allis-Chalmers,

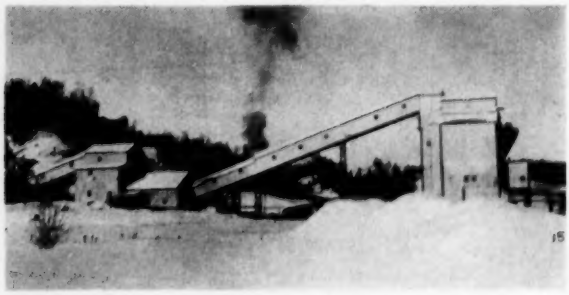


FIG. 14—CRUSHING AND MIXING PLANT

gyratory type; the feed is 14-in. and the product 6-in. There are four Allis-Chalmers 6-N gyratory crushers which reduce the 6-in. stone to 3-in. when required for Class B concrete. The product of the secondary crusher is passed through a pair of revolving screens having 2-in. circular openings which remove and waste all material smaller than 2-in. The coarse aggregate is graded from 2-in. to 6-in. By means of laboratory experiments and actual field tests it was found that coarse aggregate of this gradation will produce better concrete for less money than a uniformly graded aggregate, for the following reasons:

(a) A greater quantity of coarse aggregate can be added to a fixed amount of mortar and still produce a workable mass, than is possible with aggregate of "ideal" gradation. This amounts to a reduction of unit cement content, which means more economical concrete.

(b) It was found that any coarse aggregate ideally graded from ½ in. to 6 in., shows a pronounced tendency toward segregation

and grouping of sizes. This is especially noticeable in a 2-yd., or larger mixer. This grouping of sizes tends to harshen the mixture and interferes with workability. If the mortar content is increased until the mass becomes workable, it amounts to roughly 125 per cent of the voids in the stone.

(c) Unless the ideally graded coarse aggregate is separated into two or three sizes and then re-combined in the proper proportions, there is a very noticeable separation of sizes, or raveling, in the storage bins. This feature becomes quite troublesome due to the wide variation in gradation from batch to batch. With the smaller sizes removed, the raveling is reduced to a minimum and the workability and quality of the concrete becomes more nearly uniform.

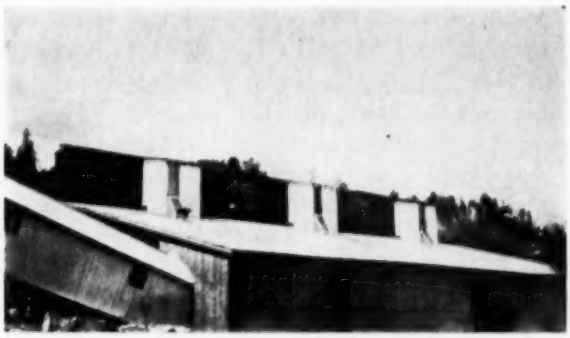


FIG. 15—CEMENT WAREHOUSE

(d) In aggregate graded from 2 in. to 6 in., a mortar content equal to about 110 per cent of the voids in the stone will produce a workable concrete.

CEMENT

Cement is purchased in bulk from the mills at Montreal and delivered to the job in daily shipments calculated to be slightly in excess of the amount required to meet the concreting schedule. The shipments range from 3 to 10 cars per day. The cars are spotted on an elevated track above the main storage bins where they are unloaded by mechanical scrapers. The actual unloading from cars to warehouse is handled by contract at a fixed price per ton—based on the railroad's weights. The mechanical unloaders

were installed and are maintained by the Aluminum company. Six screw conveyors placed below the floor level in the main warehouse bring the cement to a 30-in. belt conveyor which delivers it to a surge bin in the mixing plant building. From the surge bin it is again handled by screw conveyors to the batch weighing apparatus. At the time this paper is being written 1690 cars, containing a total of 73,962 tons of cement, have been delivered to the job. At this particular time the placing of concrete has been discontinued temporarily and the quantity of cement in storage has been intentionally reduced to a minimum in order that a check might be made. A summarized statement shows a shortage of 1.0 per cent of the total shipments, e. g., through car leakage, wind loss, handling loss, and weighing inaccuracy—1.0 per cent of the cement purchased is unaccounted for. It is safe to assume that at least one-half of this cement actually found its way into the concrete, and that the other half was direct but unavoidable loss. From experience with bulk cement in a half-million cubic yards of concrete, and regardless of the loss mentioned above, it appears to possess certain outstanding advantages over cement in bags—particularly on the large jobs. It can be unloaded faster and more economically—two men with a mechanical unloader will handle from 16 to 17 tons per hour—it permits greater flexibility in the matter of mixer-batch sizes, and there is less chance of error in the quantity going into each batch.

MIXING PLANT

The mixing plant consists of two rock storage bins—each having a total capacity of 400 cu. yds. of which about 300 cu. yds. is "live" storage; one sand storage bin having a capacity of 200 cu. yds.; two water storage tanks of 5000 gal. capacity each; two water measuring tanks of 30 cu. ft. capacity; a cement weighing device and two 4-yd. Smith mixers. The mixers have special nickel-steel linings and baffle plates to withstand the action of the unusually large coarse aggregate.

MIXTURE DESIGN

The mixture design is based on the water-cement ratio law, that is, to a designated quantity of cement and water is added sand and stone in sufficient quantity and in the most economical proportions to produce a workable mass. The workability is

varied to suit placing conditions by manipulating the sand and stone quantities rather than by changes in the cement and water content. The concrete strengths obtained by this method are parallel to, but slightly above Abram's curves.

MEASURING AGGREGATES AND MIXING CONCRETE

Sand and stone are measured by volume, cement by weight, and water by volume. The mixer measuring hoppers are of the

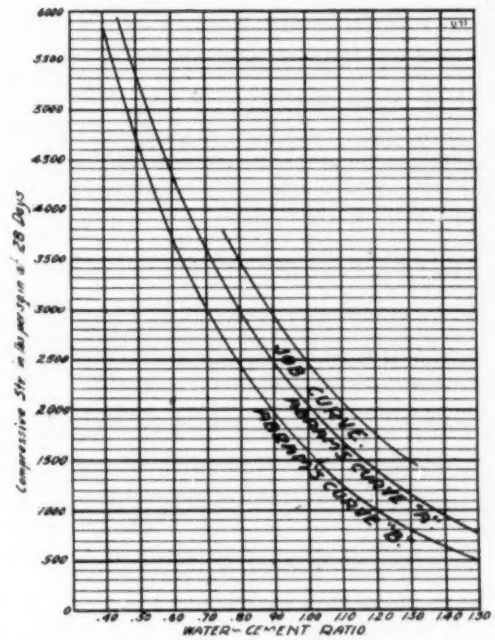


FIG. 16—JOB STRENGTH CURVE PLOTTED WITH ABRAM'S CURVES

circular type which is standard equipment for Smith mixers of this size and model. They are partitioned into two compartments for sand and stone, the cubic contents of each section being calculated per inch of depth. To the casual visitor at the mixing plant this system of proportioning seems rather inaccurate and haphazard. As a matter of fact it is recognized as being less efficient than the weighing method—nevertheless, the accuracy with which trained and experienced operators fill the hoppers to

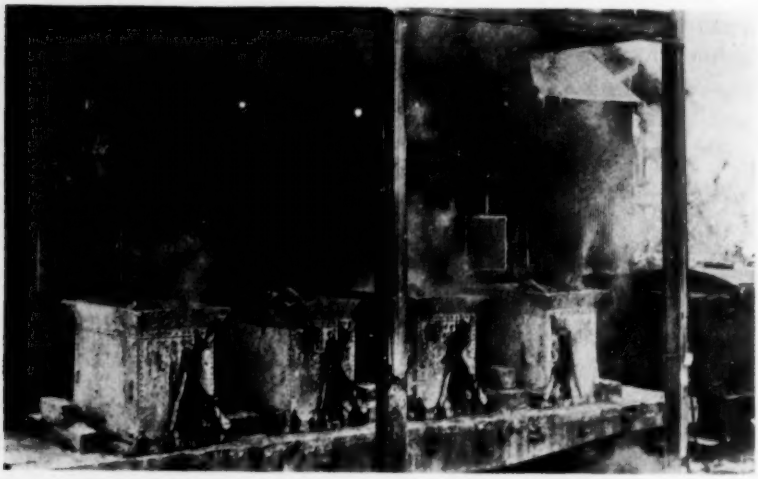


FIG. 17—4-YD. BUCKETS BEING LOADED AT PLANT

the desired point is rather astonishing. To be convinced of this, one has but to observe the uniform quantity of concrete in the 4-yd. buckets leaving the plant. These buckets are calibrated per half-inch of depth and the plant inspector checks his proportions and yield by frequently measuring the amount of concrete in the buckets. The bucket measurements are taken after the concrete reaches the forms since the movement of the train causes the concrete to settle or flatten out. Another very convincing check of the accuracy of this system of measuring the aggregates is evidenced by a comparison of the batch-count quantities with the engineers' cross-section measurement. On January 1, 1930, the total concrete placed, according to batch-count, was 286,625 cu. yds. The engineers' figures at this time showed 288,658 cu. yds., or about seven-tenths of one per cent more than the batch-count. Compared on a month by month basis, the batch-count would vary from 4.1 per cent above to 2.8 per cent below the engineers' figures. Since the engineers' monthly estimate assumes each form section to be level full instead of irregular as is actually the case, it is safe to say that the monthly batch-count is more accurate than the cross-sections.

The cement weighing apparatus consists of a Fairbanks-Morse 5200-lb. capacity platform scale equipped with a full capacity

direct reading dial, on which is mounted a weighing hopper of job design. The dial is graduated to 2 lbs. The scale is tested over its entire working range with test weights once each week and, as explained, the accuracy of the equipment is known to be within less than one per cent.

The water measuring equipment is also of job design. It consists of two 5000 gal. storage tanks which are supplied from the job reservoir. These tanks are on the hopper floor of the mixing plant building and feed by gravity to two measuring tanks at the mixers. The large storage tanks were installed to provide means

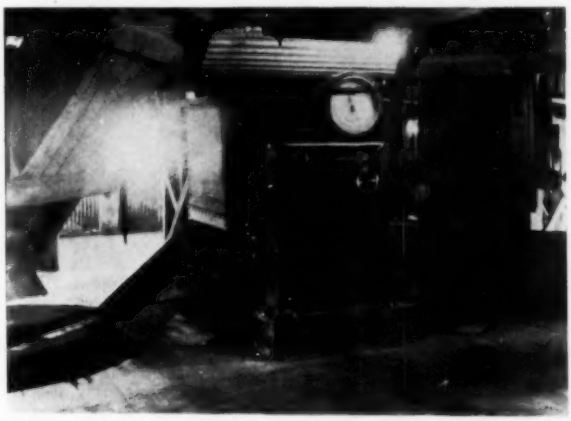


FIG. 18—CEMENT SCALE AND BATCH-HOPPER

of heating the water in the winter season. The measuring tanks are 2 ft. 3 in. in diameter and 7 ft. high and have gauge glasses graduated in cubic feet for each ½ in. of depth. Readings to each ¼ in. can be made by interpolation. The gauges are fitted with movable indicators which can be set at various positions to give the desired quantity of water for the different classes of concrete. The plant inspector signals the gauge setting from the floor above by a system of colored lights, arranged to correspond with the gauge indicators.

The mixing time is controlled by sand glasses in the hands of the assistant plant inspectors. This man spends his entire time

in a small glass-enclosed cabin, between the two mixers, and projecting far enough from the face of the main building to enable him to observe the dumping of the mixers and also to have an unobstructed view of the railroad tracks in either direction. He is in touch with the hopper floor and with both mixers by

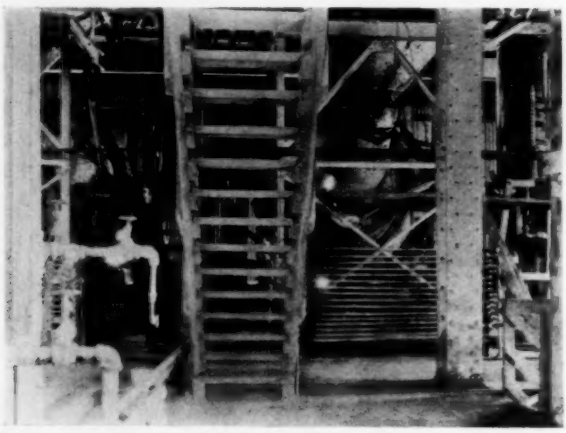


FIG. 19—REAR VIEW OF MIXERS—SHOWING WATER-MEASURING TANKS AT RIGHT

signal lights and electric buzzers. When all of the ingredients of a batch have been placed in the charging hoppers a light is flashed to the floor below. At the proper time the discharge gate is opened and the aggregates begin to flow into the mixer. When the charging hopper is completely empty a light is flashed to the assistant inspector who immediately up-ends his sandglass. When the mixing interval is complete he gives a buzzer signal to the mixer operator and the batch is discharged into the concrete buckets. The mixing time is continued for a minimum of 2-min. after all ingredients are in the drum. In cold weather, or when the working schedule permits, the batches are mixed 3-min.

In addition to timing the batches, the assistant plant inspector keeps the record of the number of batcher of each class of concrete produced and the location in which it is placed. He does this by recording the time that each train arrives at the plant, the

time required to load, the time the train departs, and the amount and class of concrete carried on the train. His record also shows the number of minutes lost each day on account of inefficient train movement, mechanical trouble at the plant, etc. It is surprising how a record of this character can be used to uncover inefficient methods and increase production. On a job of this size an accurate record of plant production is very important since it is from such a record that the Concrete Technician must study his unit material quantities and other data necessary to the proper design of the concrete mixtures. During the past several years the writer has tried out almost all types of mechanical counters, punch boards, and the various other methods of tallying, and has reached the conclusion that the method described here is about the most efficient for a job of this kind.

TRANSPORTATION SYSTEM

Concrete is handled from the plant to the forms in 2-yd. or 4-yd. buckets spaced four to the car and hauled by 40-ton dinky locomotives. The length of haul to the work on the south shore of the river is about 1 mile, and to the north shore, about 4 miles. The average running time to the north shore is 15 min., the maximum age of concrete after dumping being about 50 min. From six to ten trains are used on a normal day's run although the smaller number will scarcely take the plant capacity excepting under the most favorable conditions.

To facilitate train movement and to prevent loaded concrete trains being delayed at meeting points, etc., a dispatcher is employed to regulate the traffic. This man has an office on top of the mixing plant building from which he has a view of practically all main-line trackage and is in communication with control points by telephone. In general, the train rules specify that concrete trains are superior to all other trains, and loads have a right over empties.

The concrete buckets in use on this work are of a special design worked out by J. P. Growdon, assistant hydraulic engineer. They differ from the old type in having vertical sides and center-dump. With the stiff mixtures used it was found that buckets with sloping sides (smaller at the bottom than at the top) were quite difficult to dump and caused considerable delay which, of

FIG. 20—VIBRATING CONCRETE

course, resulted in reduced production and increased costs. The new type of bucket is regarded by those who use it as the greatest improvement in placing equipment of this kind to be developed in recent years.

PLACING METHODS

The working schedule on this project requires the use of four placing gangs. A gang consists of 1 foreman, 2 bucket-hookers, 2 bucket-dumpers, 4 vibrator-men, 2 spade-men, and 2 laborers—13 men in all. The work usually handed by a general concrete foreman is taken care of by the concrete technician and his staff of inspectors. In addition to the four placing gangs there is a small auxiliary crew composed of an electrician who services the vibrators, 3 concrete finishers, and 5 laborers who perform the various duties connected with curing the concrete. During the winter months this auxiliary crew is augmented by pipe-fitters and extra laborers.

Before describing the placing methods it will be necessary to describe the electric vibrators which are used to compact the concrete. One of these machines is shown in Fig. 20. It consists of ¼-h. p., 3-phase induction motor with an unbalanced rotor revolving at 3600 r. p. m. attached to a wooden foot which transfers the vibration to the concrete. The vibratory effect is apparent for a depth of about 16 in. in the mass. Two of these machines placed on top of a 4-yd. batch of concrete so stiff that ordinary tamping methods would make but little impression on it, will in the space of about 3-min., puddle it into a flat and compact mass, squeezing out the entrained air, and forcing the concrete into form corners and faces—leaving a smooth surface when the forms are removed.

The weight of this type of vibrator is about 160 lbs. and two men are required to handle it. There are two vibrators with each placing crew. The effectiveness of the machine is greatly increased if the operators stand on the tamping foot. For forms too small to allow the use of vibrators directly on the concrete, another type of machine—having the same size and kind of motor—is used which clamps on the studding of the form and vibrates the entire form. This type is of particular value on thin, heavily reinforced walls.

Since stiffer mixtures can be placed where vibrators are used as a means of tamping, it is estimated that, for a given water-cement ratio, the batch yield can be increased from 5 to 10 per cent—depending on the type of aggregate.

It should be emphasized that the mixtures used on this work are not so dry and harsh that they cannot be puddled properly—as a matter of fact, if the mixture were dumped in half-yard quantities it could be worked easily by the usual method of tramping. It is the size of the batch that necessitates more than ordinary treatment. If the workability of a 4-yd. batch of concrete is brought to the point where successive batches can be dumped in the forms at 3-min. intervals and properly compacted without mechanical means, the quantity of coarse and fine aggregate must be reduced sufficiently to make the mass flow easily. If this were done an accumulation of excess water would

be noticeable and the yield, as stated above, would be reduced from 5 to 10 per cent.

To provide a means of communication between the mixing plant and the forms a telephone system was installed and instruments placed at convenient points on the dam and in the power house area. The traffic over this line is principally between the placing inspectors and the mixing plant and effects changes in workability to suit different placing conditions—changes in the class of concrete being used, orders to start or discontinue mixing, etc.

No concrete is started from the plant to the forms until the placing inspector has phoned the plant that his form is ready to receive it. This means that the form-work itself has been inspected and found to be in satisfactory condition and the rock foundation or old concrete surface thoroughly cleaned and sprinkled. The first concrete to be placed in the form is a batch of grout. This consists of a mixture of sand and cement mixed in the approximate proportions of 1:2 and having a water-cement ratio of 0.80. The consistency is about the same as very thick cream. In the case of a large block in the dam, the grout is dumped in the center of the form and as near as possible to the down-stream face. It is spread in both directions, parallel to the down-stream face, by means of wire brooms and vigorously scrubbed into the old surface. After this is done the batches of class A concrete are deposited successively starting at one side of the form and extending across to the opposite side. As soon as the first batch is dumped the vibrator crews place their machines on it and compact the mass until it is flattened out to a thickness of approximately 12 in. After the down-stream face has been brought up to about one-half the depth of the scheduled lift, the crew is shifted to the up-stream face and this section is brought up in the same manner. The procedure just described leaves a valley across the center of the form between the two ridges of class A concrete. This valley is then filled with the class C mixture. The remaining portion of the lift is filled in the same manner as the first. When the form is about to be finished off, the final buckets are dumped in such a manner as to form a

series of ridges and valleys parallel to the face of the block. These corrugations are from 18 in. to 36 in. deep.

The concrete placed in the power house forms is not usually as stiff as that placed in the dam and on the greater part of it 2-yd. buckets are used. In some cases where the form is too small to permit the use of even the 2-yd. buckets, the concrete is dumped into skips, hoisted to the edge of the form with derricks and shoveled in by hand.

CURING METHODS

In all but freezing weather, all sections of freshly placed concrete are kept wet for five days. There is one exception to this rule—when a successive lift is placed on top of the first in less than five days. The wetting is done by the simple and sure method of definitely assigning certain men to keep sprinkling each new section until they are told to stop. Strange as it may seem—the sun often gets quite hot in this part of Canada, and at such times all freshly finished sections are covered with tarpaulins or burlap.

WINTER PROTECTION

It is the intention here to describe only in a general way the methods used in protecting concrete from freezing.*

The cold weather protection objective is to deliver concrete to the forms in such a condition and protect it in such a way that the temperature of no part of the mass will fall below 60° F. for at least 72 hrs. Experience on this project has shown that it is possible to realize this objective at temperatures as low as 35° below zero.

In the present season the temperature of the water in the Saguenay river, which is used for mixing, dropped from 65° F. to 55° F. from September 1 to October 1 and the mean air temperature from about 62° to 48° in the same period. The first freeze was on September 19, and the thermometer first reached the zero mark on November 23. December 5, it was 10° below zero; December 7, 20° below, and January 5, 30° to 35° below. The temperature of the river water in January was 33°.

*This subject is to be covered in detail in Mr. Burk's discussion as a critic member of Committee 604—"Winter Concreting Methods."

Protection equipment consists of two 80-h. p. boilers at the mixing plant for heating water and sand. At the forms there is one 70-h. p., one 45-h. p. and three 15-h. p. boilers and about 30 salamanders. The covering equipment consists of 200 tarpaulins, 20 ft. x 20 ft., and about 300 panels, 4 ft. x 10 ft. The panels are light wooden frames covered with canvas and roofing paper.

FIG. 21—SHOWING 4 FT. X 10 FT. PANELS ON VERTICAL SURFACE OF FORM AND CANVAS COVERING OVER THE TOP

October 1, the boilers at the mixing plant were put into operation and the mixing water heated to about 80°. This temperature was increased to a maximum of 190° as the weather got colder. On November 23, steam jets were put in the sand storage bin and stock-pile, and a little later jets were installed to discharge into the mixer drums. No attempt is made to heat the coarse aggregate.

The steam jets in the sand bin are not very effective for heating while the plant is in operation since the flow of sand through the bin is too rapid—however, they are quite useful for keeping from freezing the sand that is left in the bin over night and during periods when the plant is not in operation. The most effective method of heating sand on this project was found to be by means of steam jets placed in the cars hauling sand from the pits to the plant—and in the outdoor stock-pile. Eight 1-in. jets placed in a 30-yd. car for about 3-hrs. will thaw all frozen lumps and heat the sand to better than 100° F. An added benefit of heating in

the cars lies in the fact that the hot sand prevents the conveyor belts and screens in the screening plant from freezing. Sand will lose as much as 20° in temperature in passing from the screening plant to the mixer bins. From 10 to 15 movable jets are used for

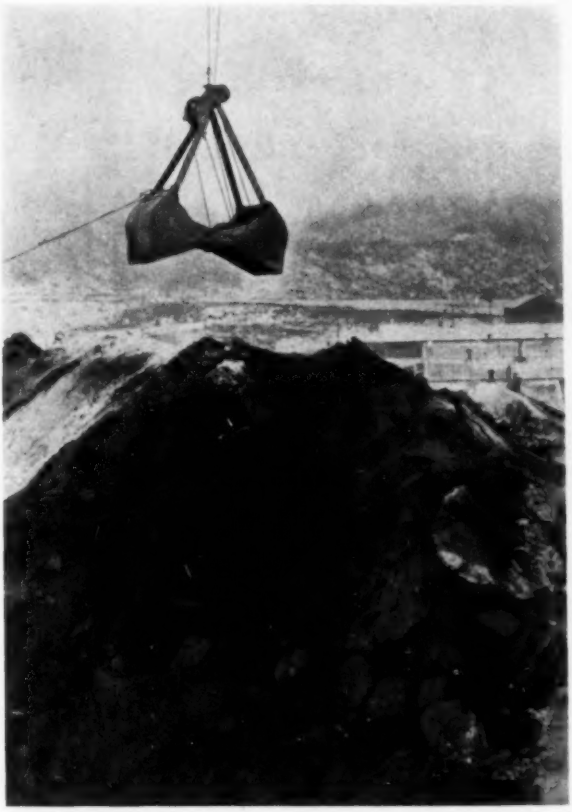
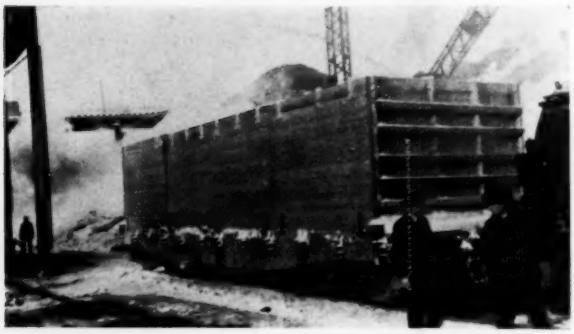


FIG. 22—FROZEN LUMPS IN SAND STOCK-PILE

thawing the frozen crust and for keeping the sand warm in the outdoor stock-pile—this storage being used only when the sand deliveries are not equal to the mixing plant requirements. The effectiveness of the stock-pile jets is increased if the sand is covered with tarpaulins.

FIG. 23—HEATING SAND IN CARS



FIGS. 24 AND 25—CAR DESIGNED TO REDUCE HEAT RADIATION DURING TRANSPORTATION

A 1-in. steam jet is arranged to discharge into the charging end of the mixer drum during the time a batch is being mixed and by means of a quick-acting valve can be cut-off while the mixer is dumping. Experiments have indicated that this live steam in the mixer will increase the temperature of the concrete as much as 10° during 3 min. of mixing.

With the heating equipment described above, the temperature of the concrete at the time it is discharged from the mixers is maintained at from 70° to 95° F. It would be possible to heat it beyond these temperatures, however, 100° F. has been designated as the maximum point to which the concrete shall be heated on this work. A temperature loss of as much as 8° has been noted between the time the mixture is placed in the buckets at the plant and its final deposit in the forms. This is the extreme, however, and represents conditions on the longest haul. In order to reduce the temperature loss on the long haul to the north shore, a special car having enclosed sides and a canvas cover was built. This car is now in experimental stage and no very definite information can be given as to its efficiency, but it is believed that it will be of considerable benefit. So the derricks may remove the buckets without damaging the car, the sides are made in hinged sections as shown in Fig. 26 and 27. This feature also allows all buckets except the one being handled by the derrick to remain under cover. The experiments under-way also contemplate the discharge of live steam, supplied by the locomotive, into the car while in transit.

Rapid transportation with no delays anywhere along the line is very necessary in cold weather and arrangements are made to have everything in readiness at the form before the concrete leaves the plant. The transportation system must be arranged in such a way that other construction trains are clear of the main tracks and sidings to be used by concrete trains; the derricks must be in proper working order and the operators "standing-by" to handle the concrete from cars to form with the least possible delay; the foundation upon which the fresh concrete is to be placed must be thoroughly thawed by steam and kept as warm as possible; and the outer sheeting on the form must be in place and the area between it and the form properly warmed by steam coils.

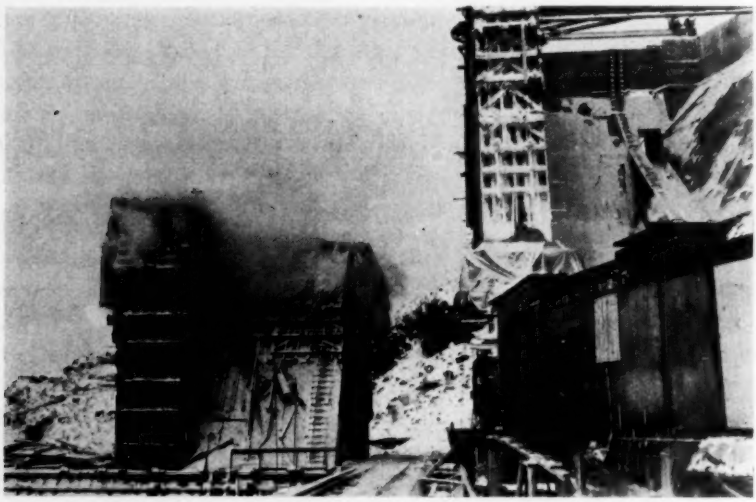


FIG. 26—EXAMPLE OF WINTER PROTECTION—THE UPPER SECTION OF A BLOCK IN THE DAM BEING CURED—AIR TEMPERATURE 20° BELOW ZERO

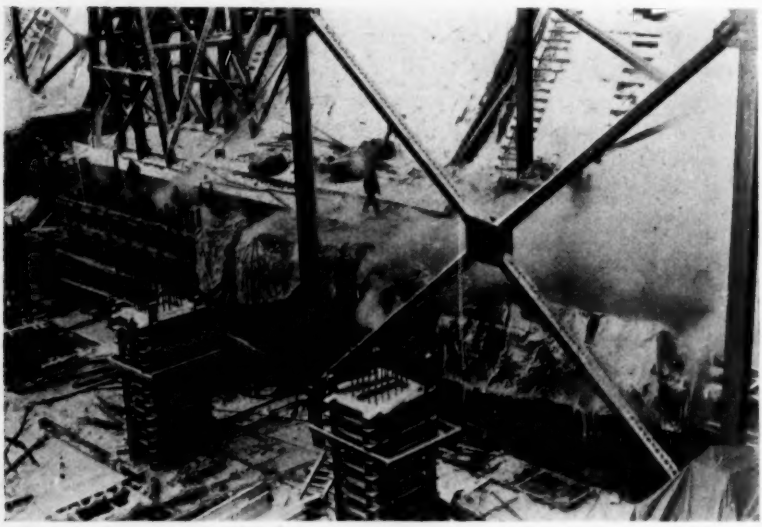


FIG. 27—PROTECTING NARROW WALLS IN THE POWER HOUSE

When everything is in readiness the actual placing of concrete proceeds in the same manner as already described.

As soon as any section of the form is filled, a rough wooden frame is built over it to support the tarpaulins—generally about 3-ft. above the concrete. The tarpaulins are then placed and steam coils put under them. Thus, the entire form is covered in a very short time after it has been filled. Special crews, directed by inspectors, are detailed (one gang during the day and one at night) to inspect the steam lines and keep the temperature within the protective enlcosure up to the desired point for at least 72-hrs. Wherever it is possible to do so without interfering with the progress of construction, forms are kept covered for 5 days.

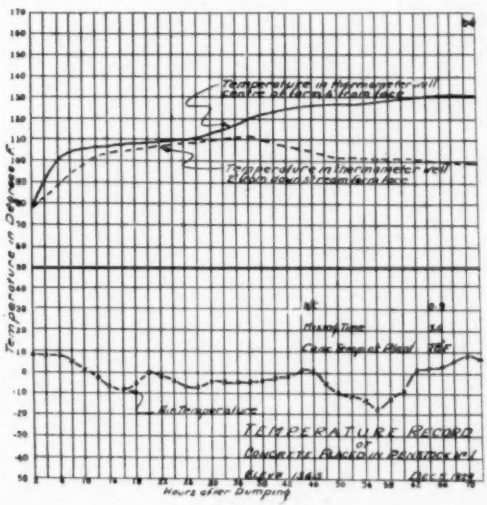


FIG. 28—CHART SHOWING RECORD OF TEMPERATURE OF CONCRETE AT VARIOUS HOURS AFTER DUMPING

As the concrete is dumped into the forms and as the placing progresses, the inspector takes frequent temperature readings in the concrete and notifies the plant if any batches show below 70° upon arrival in the forms. While the mixture is soft, thermometer wells are placed in order that a record may be kept of the temperature of the mass during the curing period. The wells are

made of ½-in. water pipe from 3 to 8 ft. long and are normally placed in three particular positions in the form; one at the up-stream face, one at the center and one at the down-stream face. Figs. 28 and 29 show a typical record of temperatures taken in this manner.

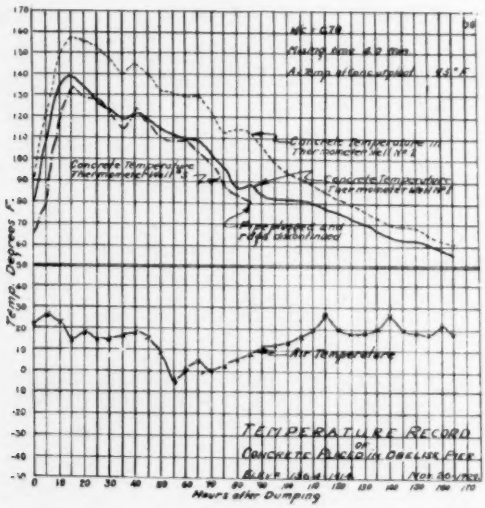


FIG. 29—CHART SHOWING TEMPERATURE OF CONCRETE PLACED IN OBELISK PIER

It is recognized that live steam, owing to the moisture supplied, provides better curing than dry coils—however, it was found necessary to arrange the piping so that both types could be used on this work. If the jets are opened while placing is still in progress, clouds of steam prevent the derrick-runners from seeing where to place the buckets, and it also becomes difficult and dangerous for the placing crews to work. Consequently, valves are placed at intervals in the coils so that dry heat is provided during the time the form is being filled, and then the valves are opened and live steam supplied when the form is finished.

INSPECTION

The inspection organization is composed of one chief inspector, two plant inspectors, one placing inspector with each concrete

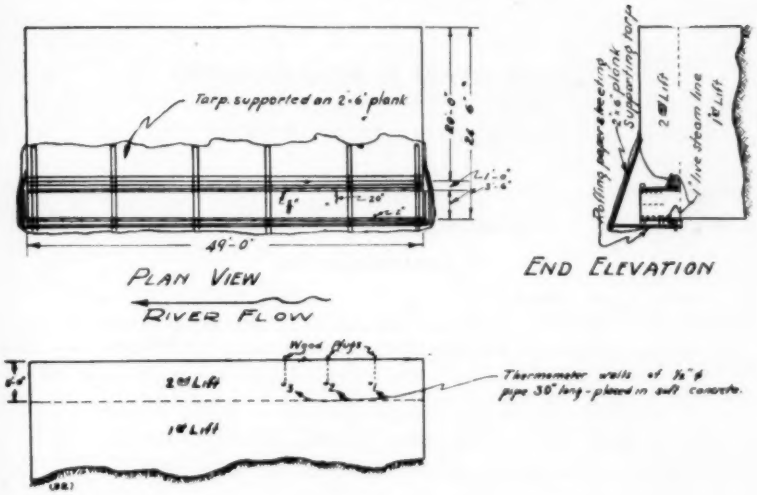


FIG. 30—SKETCH SHOWING POSITION OF THERMOMETER WELLS IN OBELISK PIER—TEMPERATURE RECORD SHOWN IN FIG. 31

gang, one materials inspector, one curing inspector (night-shift), two laboratory operators and two sample boys. The laboratory is rather well equipped and contains all of the apparatus necessary to make the usual tests of cement, aggregates and concrete.

The plant inspector has direct supervision of the proportioning workability and mixing time, and the materials inspector watches' the quality of sand and stone being produced for aggregates. The placing inspectors notify the plant as to the quality of concrete required for each form and indicate to the gang-foreman the manner in which it is to be placed and cured. The laboratory operators carry on the routine testing and make special experiments and do research work as directed.

From data furnished by the plant and placing inspectors, a daily report is compiled and furnished to the superintendent. Figs. 31 and 32 show a typical report of a day's concreting operation. In addition to showing complete data on the mixtures

SHEET No. 1

ALCOA POWER COMPANY, LIMITED

CONCRETE LABORATORY — SHIPSHAW WORKS

DAILY REPORT OF CONCRETING OPERATION

SHIFT 6:30 a.m. TO 5:15 p.m. DATE December 11th 1929

SHIFT YARDAGE 1180 WEEKS YARDAGE 3096 MONTHS YARDAGE 7020 TOTAL TO DATE 274,472

PLANT PRODUCTION

MIXER HOURS 14.5 TOTAL BATCHES 298 TIME PER BATCH 2.91 MIN.

MIXER OPERATION. DELAYS. ETC. 4.75 mixer hours waiting for trains to return from forms.
0.50 mixer hours delay due to large rock blocking gates and left mixer broken down.
0.25 mixer hours weather delay.

CLASS OF CONCRETE	NUMBER OF BATCHES	MIXTURE	WATER RATIO	YIELD (CU. YDS.)	QUANTITY CU. YDS.
Grout	8	21 : 45	.80	2.1	16.8
A	156	19 : 45 : 105	.90	4.1	639.6
B	81	24 : 45 : 85	.80	3.8	307.8
C	53	14 : 50 : 115	1.20	4.3	227.9
TOTAL PLANT YARDAGE. (BUCKET MEASUREMENT)					1192
LESS 1 PER CENT SHRINKAGE					12
ACTUAL YARDAGE CREDITED					1180

CEMENT USED 1563 BBLS. 273.45 TONS
SAND USED 674 CU YDS. 818.4 TONS
STONE USED 1088 CU. YDS. 1348 TONS
CEMENT UNLOADED AT WAREHOUSE 8 CARS

SAND HAULED { DAY SHIFT 18 CARS; NIGHT SHIFT 0 CARS
STONE CRUSHED { DAY SHIFT 56 CARS; NIGHT SHIFT 0 CARS

TEMPERATURE DATA

OUTSIDE AIR AT PLANT + 8° TO - 4° F. MIXING WATER 180 TO 190 F SAND 90 TO 90 F
CONCRETE LEAVING PLANT 76° TO 85° F INSIDE MIXING PLANT 10° TO 30° F.

REMARKS: SAND AVERAGE:
Moisture - 6.0%
Bulking - 33.0%
Wt. loose wet 90.0#
Clay and Silt (Car Sample) - - 2.5%
Clay and Silt (Mixer Hopper Sample) 3.1%
Organic Matter Color - (CLEAR)

CEMENT AVERAGE:
Class A 436# per cu. yd.
" B 504# " " "
" C 306# " " "
Grout 940# " " "

C. D. Durham INSPECTOR

FORM LAB. No. 3 2/29 SH.

FIG. 31—PLANT INSPECTOR'S REPORT

used, this report also indicates the location of, and quantity of concrete placed in each form that day.

TESTS AND EXPERIMENTS

The daily laboratory routine consists in testing the cement and

SHEET NO 2

ALCOA POWER COMPANY, LIMITED

CONCRETE LABORATORY — SHIPSHAW WORKS

DAILY REPORT OF CONCRETING OPERATION

PLACING RECORD

SHIFT 6:30 a.m. TO 5:15 p.m. DATE December 11th 1929

LOCATION PLACED	CLASS OF CONCRETE	No. OF BATCHES	YIELD (CU. YDS.)	BUCKET YARDAGE	LESS 1 % SHRINKAGE	TOTAL CREDITED YARDAGE
Block 17N Main Dam elev. 205 to 213 H. Porter - Insp.	Grout	2	2.1	4.2		
	A	104	4.1	426.4	4.3	
	C	53	4.3	227.9	2.3	652
Power House Generator Floor #2 to elev. 123 P. M. Fenton - Insp.	Grout	5	2.1	10.5		
	B	61	3.8	307.8	3.1	315
Block 14N Pier elev. 210 to elev. 220 J. L. Schaffner - Insp.	Grout	1	2.1	2.1		
	A	58	4.1	213.2	2.1	213
		298				1180

TEMPERATURE DATA

OUTSIDE AIR 4 TO -4 F. CONC. ARRIVING AT FORMS 70 TO 80 F. CONC. AFTER DUMPING 65 TO 74 F.

WEATHER CONDITIONS Cloudy with moderate North wind.

REMARKS: Held up 10-min. on 17N to thaw out ice in one corner of old surface.

FORM LAB. NO. 28 2/29 5M. T. M. Nevin INSPECTOR

FIG. 32—PLACING INSPECTOR'S REPORT

sand, making and breaking concrete compression specimens, posting the laboratory records and compiling reports.

All physical tests are made on cement in conformity with A. S. T. M. standard specification C 9-26—with the exception that only one composite sample is tested daily. This composite sample is made up from grab samples taken hourly from the weighing hopper. Unless there is reason to suspect that the cement does not meet specification requirements, shipments are not held up until test results are available. On previous projects of this type the writer has used the method of having special bins set aside at the mill and samples taken as these were filled and then the bins sealed until test results were available—he has also held bag cement in job warehouses until 7-day strengths had been made. With the exception of a slight variation in fineness on rare occasions, ten years experience has indicated that cement produced at recognized mills is invariably above standard specification requirements. In any event, if it is thought necessary to pre-test cement, probably the most convenient and satisfactory method of doing it would be for the purchaser to station a competent man at the mill laboratory—but, in the writer's opinion, this is seldom necessary. The cement tests made at Chute a Caron Dam are principally for study in connection with concrete strengths. Test results averaged for the past several months show the characteristics of the cement to be as follows:

Setting Time (Gilmore)	Tensile Strength (1:3 Mortar Briquettes)
Initial: 2-hr. 50-min.	7-days 286
Final: 6-hr. 10-min.	28-days 357
Normal Consistency 23.0	Compression Strength
Soundness: O. K.	(1:3 mortar—2″ x 4″ cyls.)
Fineness: 17 per cent	7-days 2087
	28-days 2884

In addition to visual inspection of the sand being loaded at the pits, test samples are taken from each car delivered to the screening plant. These samples are taken with a sand riffle at the point where the finished sand discharges into the stock-pile and tests are made on it for silt and clay and organic matter. A mechanical analysis is made twice daily on composite samples. The moisture content, bulking, and weight per cubic foot are checked hourly by samples taken from the discharge chute of the plant sand bin—these tests being made to enable the plant inspector to calculate

the amount of free water carried by the aggregate. The weight per cubic foot must be based as nearly as possible on the actual condition of the sand in the measuring hopper, and this figure is obtained by placing a standard cubic foot measure about halfway down in the hopper and allowing the sand to fall into it in the same manner that the hopper itself is filled.

Concrete samples for compression tests are taken from the buckets at the mixing plant and brought to the laboratory where all particles of coarse aggregate are removed from the mass and the remaining mortar formed into 6 in. x 12 in. cylindrical test specimens. One group of six specimens is taken to represent each

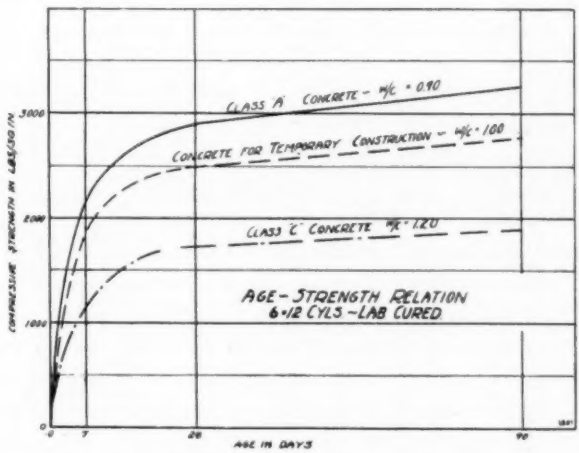


FIG. 33—SHOWING AGE STRENGTH RELATION

class of concrete being produced each day. Specimens are cured under standard conditions and broken at the 7, 28 and 90-day periods. Since the concrete contains particles of coarse aggregate as large as 6 in., it could not, of course, be formed into specimens small enough to be tested in a compression machine of 200,000 lbs. capacity, which is the size machine normally used in field laboratories. If an attempt is made to remove from the mass

CONCRETE LABORATORY. ALCOA POWER COMPANY, LIMITED.

Lab. No.	Date Made	Description of Sample	Mixture Proportions (Cu. Ft.)				Water Ratio	F. M. Sand	F. M. Stone	Yield (Cu. Ft. Per Mixed Batch)	Cement Per Cu. Yd. (Lbs.)	Test Specimens		
			Cement	Sand	Stone	Water						No. Made	Size and Shape	Slump
	1928													
775	Oct. 1		13	47	97	16.9	1.3	1.93	9.5	106.7	309	6	6"x12" Cyl.	5"
782	2		13	47	103	16.9	1.3	2.08	9.5	108.3	305	6	6"x12" Cyl.	5"
783	3		19	47	95	17.1	0.9	2.08	8.8	102.6	470	6	6"x12" Cyl.	2"
789	3		19	47	97	17.1	0.9	2.29	8.8	106.9	451	6	6"x12" Cyl.	2½"
790	3		13	47	100	17.1	1.3	2.29	9.5	111	298	6	6"x12" Cyl.	6"
797	4		20	47	95	18	0.9	2.28	8.8	108	470	6	6"x12" Cyl.	5"
798	4		14	47	100	12.6	0.9	2.28	9.5	109.6	324	6	6"x12" Cyl.	10½"
806	5		14	47	103	18.2	1.3	2.70	9.5	114.2	311	6	6"x12" Cyl.	10"
807	5		20	47	100	18	0.9	2.70	9.5	108.8	467	6	6"x12" Cyl.	7"
817	6		14	47	103	18.2	1.3	2.59	9.5	114	311	6	6"x12" Cyl.	7"
818	6		22	47	97	19.8	0.9	2.59	8.8	105.8	528	6	6"x12" Cyl.	10½"
825	8		20	47	100	18	0.9	2.31	9.5	114.2	445	6	6"x12" Cyl.	5½"
828	8		14	47	103	18.2	1.3	2.31	9.5	107.2	332	6	6"x12" Cyl.	7"
835	9		20	47	100	18	0.9	2.41	9.5	107.5	472	6	6"x12" Cyl.	7½"
836	9		14	47	103	18.2	1.3	2.41	9.5	114	312	6	6"x12" Cyl.	8"
842	10		20	47	100	18	0.9	2.78	9.5	101.9	475	6	6"x12" Cyl.	4"
843	10		14	47	103	18.2	1.3	2.78	9.5	113.4	313	6	6"x12" Cyl.	9"
851	11		22	47	100	19.8	0.9	2.22	9.5	108.5	514	6	6"x12" Cyl.	10"
853	11		14	47	103	18.2	1.3	2.22	9.5	113.9	313	6	6"x12" Cyl.	5½"
861	12		20	47	100	18	0.9	2.14	9.5	108	470	6	6"x12" Cyl.	2½"
862	12		14	47	103	18.2	1.3	2.14	9.5	114.2	311	6	6"x12" Cyl.	9"
868	13		24	47	100	21.6	0.9	2.28	9.5	108	564	6	6"x12" Cyl.	9"
876	15		24	47	100	21.6	0.9	2.31	9.5	108.9	558	6	6"x12" Cyl.	10"
883	16		24	47	100	21.6	0.9	1.91	9.5	108	564	6	6"x12" Cyl.	11"
884	16		14	47	100	18.2	1.3	1.91	9.5	110.7	321	6	6"x12" Cyl.	6"

Cone, sample taken from form immediately after dumping. Coarse agg. above ¾" removed by hand picking. Cured at Lab. under standard condition

*Typical Monthly Consolidated Strength Report.

SHIPSHAW WORKS. RECORD OF COMPRESSION TESTS ON CONCRETE SPECIMENS.*

COMPRESSION STRENGTH										Estimated Strength Of Concrete At 28 Days		Location in Which Concrete Represented By These Specimens Was Placed	REMARKS	Lab. No.
Age 7 Days			Age 28 Days			Other Ages								
Weight	Total Load	Unit Load	Weight	Total Load	Unit Load	Age	Weight	Total Load	Unit Load	% of Mort.				
27.6	46000	1630	27.4	65000	2297	198 Dys.	26.9	84500	2980					
27.3	37000	1310	27.9	45000	1590	198 Dys.	27.6	88000	3110	65	1260	Block 5-N to Elev. 170		775
27.3	27000	954	27.6	44000	1554	205 Dys.	27.9	44000	1554					
27.7	28000	989	27.6	41000	1448	204 Dys.	27.9	30000	1060	65	980	Block 12-N to Elev. 159		782
28.5	75000	2650	28.9	103000	3640	205 Dys.	28.8	136000	4805					
28.3	86000	3039	28.0	91000	3215	205 Dys.	28.4	140500	4968	85	2920	Block 5-N to Elev. 177		783
27.3	87000	3074	27.0	93000	3283	196 Dys.	27.5	155000	5480					
27.3	88000	3109	27.2	100000	3533	196 Dys.	27.4	140000	4950	85	2900	Block 5-N to Elev. 177		789
27.4	48000	1696	27.8	57000	2014	196 Dys.	27.6	110000	3900					
27.6	55000	1940	27.1	71000	2510	196 Dys.	28.0	97500	3450	65	1470	Block 5-N to Elev. 177		790
27.5	80000	2826	28.1	108000	3816	202 Dys.	28.3	105000	3710					
21.1	84000	2970	27.6	120000	4240	202 Dys.	27.7	124500	4405	85	3430	Block 5-N to Elev. 180		797
28.0	48000	1696	28.5	77000	2720	203 Dys.	27.9	113000	3996					
27.9	47000	1660	28.0	75000	2650	201 Dys.	28.0	82000	2948	80	2150	Block 5-N to Elev. 180		798
27.7	45000	1590	27.7	68000	2400	195 Dys.	28.0	107500	3800					
38.0	46000	1625	28.3	75000	2650	195 Dys.	27.4	90000	3180	65	1640	Block 12-N to Elev. 165		806
27.9	78000	2756	27.9	111000	3920	195 Dys.	28.1	135000	4770					
28.6	70000	2470	28.4	105000	3710	195 Dys.	28.2	123000	4350	80	3050	Block 18-N to Elev. 170		807
			28.3	72000	2544	193 Dys.	27.9	106000	3740					
28.3	47000	1660	27.9	71000	2510	194 Dys.	28.0	102000	3610	65	1640	Block 18-N to Elev. 173		817
			28.4	99000	3500	194 Dys.	28.2	115000	4070					
27.8	57000	2010	28.2	87000	3074	194 Dys.	28.4	141000	4980	85	2800	Block 12-N to Elev. 170		818
			28.4	109000	3851	199 Dys.	28.3	134500	4752					
28.4	67000	2370	28.1	98000	3463	198 Dys.	28.3	139000	4915	80	2930	Block 5-N to Elev. 189		825
			27.5	60000	2120	197 Dys.	28.4	84500	2986					
28.4	37000	1307	28.3	65000	2297	198 Dys.	28.2	79000	2790	65	1440	Block 18-N to Elev. 178		828
			28.3	100000	3533	191 Dys.	28.2	120000	4240					
28.3	73000	2580	28.1	95000	3360	191 Dys.	28.2	117500	4150	80	2760	Block 18-N to Elev. 183		835
			28.2	68000	2403	191 Dys.	27.9	80000	2830					
28.5	50000	1770	28.6	79000	2800	191 Dys.	28.0	110000	3890	65	1690	Block 18-N to Elev. 183		836
			29.0	100000	3533	196 Dys.	28.9	120500	4257					
28.3	81000	2863	28.4	105000	3710	196 Dys.	28.8	124000	4387	80	2900	Block 12-N to Elev. 175		842
			28.1	58000	2050	197 Dys.	28.5	88000	3109					
28.3	49000	1731	28.1	67000	2370	197 Dys.	27.6	89500	3165	65	1440	Block 18-N to Elev. 183		843
			28.1	120000	4240	189 Dys.	28.0	95000	3360					
28.4	77000	2720	28.6	117000	4134	189 Dys.	27.8	145000	5120	80	3350	Block 5-N to Elev. 197		851
			28.4	75000	2650	190 Dys.	28.1	100000	3540					
28.6	50000	1770	29.0	75000	2650	190 Dys.	28.4	114500	4050	65	1720	Block 12-N to Elev. 180		853
			28.6	120000	4240	188 Dys.	28.2	135000	4770					
28.7	78000	2756	28.4	119000	4205	189 Dys.	28.3	135500	4790	80	3380	Block 18-N to Elev. 192		861
			28.3	70000	2470	189 Dys.	28.0	100500	3550					
28.6	48000	1696	28.2	71000	2510	189 Dys.	28.5	103500	3660	65	1620	Block 18-N to Elev. 192		862
			28.1	112000	3958	187 Dys.	28.1	120000	4240			By Pass Wall 125	North	
28.1	80000	2826	28.8	100000	3533	187 Dys.	28.0	140000	4950	80	3000	St. 17 + 77 — 18 — 40	Side	868
			28.1	116000	4100	192 Dys.	28.6	135000	4770					
28.8	77000	2720	28.0	119000	4205	190 Dys.	28.9	121500	4290	80	3320	Block 12-N to Elev. 189		876
			27.8	76000	2865	184 Dys.	27.6	115000	4060					
28.1	62000	2190	28.1	98000	3463	184 Dys.	28.0	147000	5200	80	2520	Block 18-N to Elev. 203		883
			27.9	68000	2403	184 Dys.	28.8	87500	3100					
28.3	41000	1448	28.5	69000	2438	185 Dys.	28.0	95500	3370	65	1570	Block 18-N to Elev. 203		884

205 and 204 Day specimens had defective ends

ALCOA POWER COMPANY LIMITED

CONCRETE LABORATORY—SHIPSHAW WORKS

COMPRESSION SPECIMEN DATA

MADE BY. E.V.H. DATE October 3rd 1928 TIME 11:00 a.m.

MATERIAL FOR SAMPLE WAS TAKEN FROM Buckets at Mixing Plant.

Class A concrete being used in Block 5W Elev. 177

MIXER BATCH

CEMENT 19 CU. FT. 1786 LBS.
SAND 70 CU. FT. 5610 LBS.
STONE 97 CU. FT. 8250 LBS.
WATER 12.9 CU. FT. 96.6 GAL.
BATCH YIELD (WET) 106.9 CU. FT
CEMENT PER CU. YD. OF CONCRETE 451 LBS.
MIXING TIME 2 MIN. 11 REV'L per min.

MATERIALS

SAND: MOISTURE 4.58 %. BULKING 33 % F.M. 2.29
STONE Product of #15 & 6-K Crushers F.M. 6.8
BRAND OF CEMENT USED Canada Cement Company
APPEARANCE OF MIXTURE Slump 2½-in.
Mix appeared plastic and easy to work.

TEMPERATURE DATA

MIXING WATER 112-152 F SAND 50-56 F CONC. AT PLANT 70° F AR. AT FORMS 69° F
IN PLACE 67° F 24 HRS. AFTER PLACING 110° F WEATHER Clear and warm
REMARKS: Unit weight of sand 63# (loose wet)

TEST RESULTS

NUMBER AND SIZE OF SPECIMENS MADE 6 - 6" x 12" cylinders GROUP LAB. No. 709
DESIGNED STRENGTH 3000 LBS.SQ. IN. WATER RATIO 0.9 PROBABLE STR. OF CONC. 2900 LBS.SQ. IN.

SPEC. No.	DATE BROKEN	AGE	END AREA	WEIGHT	TOTAL LOAD	UNIT LOAD	TYPE OF BREAK
1	Oct. 10,1928	7-days	28.27	27.312	87,000	3074	Cone
2	Oct. 10,1928	7-days	28.27	27.312	88,000	3109	Cone
3	Oct. 31,1928	28-days	28.27	27.0	93,000	3288	Cone
4	Oct. 31,1928	28-days	28.27	27.187	100,000	3533	Cone
5	Apr. 17,1929	196-days	28.27	27.5	155,000	5480	Cone
6	Apr. 17,1929	196-days	28.27	27.375	140,000	4950	Split
7							
8							

FORM LAB. No. 2 2/28 5M.

FIG. 34—TYPICAL REPORT OF COMPRESSION SPECIMEN DATA
TYPICAL MONTHLY CONSOLIDATED STRENGTH REPORT

only those stone sizes which are too large for the molds to be used, then the quantity of stone remaining in the mass may vary over a range sufficient to give very eratic test results. This situation

actually developed in the construction of the Santeetlah Dam (1926-27). Since the necessity of changing the composition of the test-mass to *some* extent was recognized, it seemed that the logical thing to do was to remove *all* coarse aggregate from the mass and simply test the mortar. This brought up the question of the relation of mortar-strength to concrete-strength. A special test was devised which promised to throw some light on the matter. Test cylinders 3 ft. in diameter by 6 ft. high were cast using the unaltered mass of concrete just as it came from the mixers, and from the same batch 6 in. x 12 in. specimens were made composed of mortar only. The series included several different water ratios and enough specimens were made to fill three standard railroad gondola cars. These were shipped to the National Bureau of Standards and tested at the approximate age of 90 days. From the results it was found that the 6 in. x 12 in. mortar specimens gave strengths equal to from 115 to 135 per cent of the 3-ft. x 6-ft. cylinders. While the results were not conclusive in many respects, the data obtained is all being used as a means of estimating the "actual" strength of concrete where the strength of its mortar, as indicated by 6 in. x 12 in. specimens, is known. In determining the probable strength of concrete from the known strength of 6 in. x 12 in. mortar cylinders made from the same mixture, a correction factor is applied. This factor is not a constant but is affected by the water-cement ratio of the mixture and the relative quantity of coarse aggregate. For example, a mixture having a water-cement ratio of 1.3 with the maximum quantity of coarse aggregate, would be considered as representing the widest gap in the mortar-concrete strength relation and the mortar-strength would be assumed to represent 135 per cent of the concrete-strength. The opposite extreme would be a mixture having a water-cement ratio of 0.80 and being slightly over-sanded. In this case the mortar-strength would be taken as indicating 115 per cent of the concrete-strength. The correction factors have been applied to the "Job Curve" shown in Fig. 16 and to the strengths indicated in Fig. 33.

Fig. 34 shows a typical report on a single group of compression specimens and the strength table shows the consolidated monthly report of the mixtures used and strengths obtained. During the 1928 season the average quantity of cement per cubic yard for

the various classes of concrete was as follows: Class A, 503-lbs.; class B, 572-lbs.; class C, 336-lbs.; Grout, 989-lbs. The average of all classes was 461-lbs.

CONCLUSION

The type of field inspection and the methods of mixture design and control described in the foregoing pages had not interferred to any appreciable extent with the progress of construction and the cost is not much above the sum usually paid to outside concerns for testing the cement alone. It is needless to say that the success of inspection of this character depends to a very large extent upon the attitude of the chief engineer and the cooperation of the job superintendent. On this project, Mr. Rickey and his assistants have not only encouraged the organization by frequent inspection trips to the job, but have contributed valuable suggestions and advise. Mr. White, superintendent, has given his full cooperation and assistance at all times.

Readers are referred to the JOURNAL *for June, 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Study of Methods of Curing Concrete

BY H. F. GONNERMAN*

SUMMARY

Studies by numerous investigators have conclusively demonstrated that adequate curing of concrete is essential in order to develop to a high degree the desirable properties of concrete. With lack of curing the development of strength and resistance to wear are greatly retarded. Other properties are likewise adversely affected by lack of curing and recent investigations have shown that curing becomes of particular importance where watertightness† and durability under severe conditions of exposure are desired.

Curing with water has long been recognized as the most satisfactory method but under some conditions it is difficult to secure adequate water curing on account of cost or scarcity of water and as a consequence new methods are promoted from time to time. This laboratory investigation was undertaken in order to obtain information on the relative effectiveness of several of the methods recently advocated for curing concrete roads. Tests were made to study the effect of these methods on the compressive and flexural strength, wear and surface hardness of concrete, controlling as carefully as possible all the variables that might influence the results.

The test specimens consisted of 6 by 12-in. compression cylinders, 7 by 7 by 38-in. beams for flexure tests and 8 by 8 by 5-in. blocks for wear and absorption tests. The concrete used in the tests was of a quality ordinarily employed in road construction.

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†"Some Permeability Studies of Concrete", by F. R. McMillan and Inge Lyse. Journal of the American Concrete Institute, December, 1929, p. 101.

The tests included 17 different methods grouped into four general classes as follows:

(1) *Water Curing*

Specimens either in water, moist room, or in air at 50 per cent humidity as indicated below, the temperature in all cases being maintained at 70° F.

(a) Molds 1 day covered with wet burlap, in water until test.

(b) Molds 1 day covered with wet burlap, moist room 2, 6 or 13 days, in air until test. One group of compression cylinders cured in moist-room entire time before test.

(c) Molds 1 day not covered, in air until test.

(d) Molds 1 day covered with wet burlap, left in molds uncovered in air until test.

(2) *Surface Coatings*

(a) McEverlast paint (Hunt Process) and Curcrete sprayed on top surface of specimens immediately after molding; other surfaces sprayed on removal of molds at 24 hr. and specimens stored in air at 50 per cent humidity until test. For one group of specimens top surfaces sprayed with McEverlast paint and specimens left in molds in air at 50 per cent humidity until test.

(b) Molds 1 day covered with wet burlap. Upon removal of molds specimens painted on all sides with Tarvia K. P., sodium silicate, paraffin and raw linseed oil and then stored in air at 50 per cent humidity until test.

(c) Molds 1 day covered with wet burlap. Upon removal of molds specimens cured in Sisalkraft paper bags in air at 50 per cent humidity until test.

(3) *Calcium Chloride Admixture—2 per cent by Weight of Cement*

(a) Molds 1 day covered with wet burlap, then in water at 70° F. until test.

(b) Molds 1 day not covered, then in air of laboratory at 50 per cent humidity until test.

(4) *Calcium Chloride Surface Covering—2½ lb. per sq. yd.*

(a) Molds 1 day covered with wet burlap, top surfaces then covered with calcium chloride and specimens left in molds in air at 50 per cent humidity until test.

An important feature of the investigation was the use of duplicate sets of strength specimens for each method of curing for test at ages of 1 day to 1 year. One set was tested "as cured" and the other was tested after soaking in water in order to bring the specimens to the same (or greater) weight as when removed from the molds, thus eliminating the factor of variable moisture content at time of test.

Conclusions, so far as the strength tests are concerned, are based on the results obtained with specimens tested in a saturated condition. The compressive strengths are given most weight

because the compression cylinders were, in general, more thoroughly saturated than the beams after soaking in water for the periods used.

In applying the results of these tests to the curing of concrete pavements the possible influence of subgrade conditions on the effectiveness of surface coverings and coatings should be given consideration. Removal of water from the concrete by a dry, absorbent subgrade might impair the effectiveness of these methods, while a thoroughly wetted subgrade or a paper covering on the subgrade might be expected to be of advantage. The effects of increase in temperature of the concrete due to the absorption of heat where bituminous coatings are used should also be given consideration.

The principal conclusions from the tests are as follows:

(1) There were marked differences in the strength of similar specimens cured in the same manner but tested wet on the one hand and "as cured"—that is, dry or semi-dry on the other.

Cylinders tested wet always showed lower strengths than when tested "as cured" except in a few cases at early ages when about the same strengths were obtained. Where cylinders were cured by means of surface coatings the differences in strength of specimens tested wet and "as cured" generally became more pronounced after about the 28-day period while for many of the other methods these differences became appreciable at earlier ages. For certain methods, cylinders tested wet had only 75 or 80 per cent of the strength of similarly cured cylinders tested "as cured."

In contrast to the cylinders, beams tested wet were generally stronger at all ages than beams tested "as cured." A few exceptions occurred in the 1-yr. tests. The modulus of rupture of beams tested wet and "as cured" appeared to be greatly influenced by the extent to which the beams had dried out and the degree of saturation when tested.

(2) Cylinders cured by the various methods showed only minor differences in compressive strength at ages of one to seven days, most of them having strengths equal to or greater than water-cured cylinders at these ages. At later ages, the differences in strength for the various methods became more marked and at 3

months and 1 year none gave strengths equal to continued water curing.

An outstanding feature of the results obtained in tests of specimens cured in moist room for periods up to 13 days was the gradual improvement in both compressive and flexural strength at a given age with increase in the duration of moist curing.

Cylinders cured in Sisalkraft bags or by means of Tarvia, McEverlast paint, Curcrete, linseed oil and paraffin had approximately the same strengths at the different ages as cylinders cured *in moist room for 13 days*. Compressive strengths for these methods ranged from 75 to 85 per cent at 3 months and from 58 to 76 per cent at 1 year of the strength of cylinders cured continually in water.

Lowest compressive and flexural strengths were obtained with air-cured specimens with and without admixture of calcium chloride, and with specimens coated with sodium silicate. For these methods the strengths at 1 year were about half those of specimens cured continually in water, and about 70 per cent of those cured 13 days in moist room. From these results it would appear that the coating of sodium silicate was of little or no value as a means of curing concrete, also that the admixture of calcium chloride, while increasing the early strength, was not a satisfactory substitute for a few days of moist curing.

INTRODUCTION

CURING is one of the most important factors affecting the performance of concrete in service especially concrete exposed to severe climatic conditions, to excessive wear or to water under pressure. With the development and promotion of new methods of curing concrete, particularly concrete roads, there is need for information on the effectiveness of several of the methods now being advocated. It is generally recognized that concrete of the consistency used in highway work contains more water than is necessary for proper curing so that if excessive loss of water through the subgrade, or by evaporation, is prevented during the early hardening period the concrete will continue to gain in strength for a considerable time.

Among the methods commonly employed for curing roads are, ponding or covering with wet earth, hay or straw kept wet for

periods up to 14 days and calcium chloride as a surface coating. An admixture of calcium chloride is sometimes used to increase early strength and is claimed to obviate the necessity for relatively long periods of moist curing. In the last few years the use of surface coatings of sodium silicate and asphaltic materials have been actively promoted for curing concrete roads.

While a great number of field tests on curing have been made throughout the country, the information thus far obtained has not generally been conclusive due to lack of control of factors which are known to have an appreciable effect on the results.† Some of the more important of these factors are (1) temperature, (2) humidity and rainfall, and (3) moisture content and temperature of specimens at time of test.

The flexural tests, in general, confirmed the results of the compression tests, rating the various curing methods in the same order. Because of the difficulty encountered in thoroughly saturating specimens cured by the better methods, further studies using beams of various sizes, different methods of saturation and periods of immersion are recommended in order to develop a satisfactory technique of testing.

(3) The results of the wear tests checked the strength tests and emphasized the harmful effect of lack of curing or insufficient curing on the wearing resistance of the concrete. Air-cured concrete with and without admixture of calcium chloride, concrete cured in moist room for 2 days and concrete cured with sodium silicate had relatively low resistance to wear while concrete cured by the other methods had approximately the same resistance to wear as concrete cured moist for 13 days.

(4) Measurements of the amount of moisture lost from the specimens during curing showed that those treated with the bituminous, linseed oil, and paraffin coatings and those encased in Sisalkraft paper lost water gradually but at a greatly retarded rate as compared with the rapid loss for the air-cured and sodium silicate coated specimens. In general, the specimens showing greatest losses of moisture at time of test had lowest strengths.

(5) Absorption tests on 8 by 8 by 5-in. concrete blocks at age of 3 months confirmed the strength and wear tests in that they

†See Report of Special Committee of Highway Research Board on "Curing," *Proceedings of the Highway Research Board, 1929.*

showed lowest absorptions for the curing methods which developed greatest strength and resistance to wear.

(6) While the ball indentation test was not sufficiently accurate to definitely establish the relative surface hardness of concretes having about the same strengths, nevertheless it showed consistent differences in surface hardness of concretes cured by the best and the poorest methods as judged by the results of the strength and wear tests.

ACKNOWLEDGMENT

Acknowledgment is made to Albert Timms, Associate Engineer, and Wm. R. Johnson, formerly Assistant Engineer of the laboratory staff and now employed by the Aluminum Company of America as concrete technician on the Calderwood Dam, for their valuable assistance in compiling and preparing the data used in the preparation of this paper.

PURPOSE AND SCOPE OF TESTS

The purpose of this series of tests was to study the effect on the compressive and flexural strength, wear and surface hardness of concrete of some of the different methods of curing now in use, controlling as carefully as possible all the variables that might influence the results. The tests included 17 different methods which may be grouped into four general classes. All specimens, unless otherwise stated, were stored indoors in a room maintained as closely as possible at a temperature of 70°F. and a relative humidity of 50 per cent.

I. *Water Curing:* All specimens in molds 1 day, covered with wet burlap (except No. 5) then, either in water or moist room or in the air at 50 per cent humidity as indicated below. In all cases, temperature was maintained at 70°F.

(1) Molds 1 day, in water until tested.
(2) " " " moist room 13 days, air until test.
(3) " " " moist room 6 days, air until test.
(4) " " " moist room 2 days, air until test.
(5) " " " not covered, then in air until test.
(17) " " " covered with wet burlap, then left in molds uncovered in air until test.

II. *Surface Coatings:* In methods 6, 7, and 9, top surfaces of specimens sprayed with coating material immediately after finishing, but no other protection during first day. Other sur-

faces coated upon removal of molds except in method 7 where specimens were left in molds until tested.

In methods 8, 10, 11, 12 and 13, specimens kept covered first day with burlap and coating applied to all surfaces upon removal of molds.

In all cases except method 7, molds removed at 1 day and specimens placed in air at 50 per cent humidity until test.

(6) Hunt Process, spray coat of McEverlast paint.
(7) Hunt Process, spray coat of McEverlast paint on top only, left in molds until tested.
(8) Tarvia K. P., 1 coat applied with brush.
(9) Curcrete, spray coat.
(10) Paraffin, applied hot with brush.
(11) Sodium silicate (36 or 37° Baume), 3 brush coats.
(12) Raw linseed oil, 3 coats applied cold with brush.
(13) Sisalkraft paper bags placed around specimens when molds were removed.

III. *Calcium Chloride Admixture*—2 per cent by weight of cement.

(14) In molds 1 day uncovered, then forms removed, and stored in air at 50 per cent humidity until tested.
(15) In molds, 1 day under wet burlap, then stored in water until tested.

IV. *Surface Covering, Calcium Chloride*—2½ lb. per sq. yd.

(16) In molds 1 day, covered with wet burlap, then top surface covered with calcium chloride, specimens left in molds in air at 50 per cent humidity until tested.

MATERIALS AND TEST METHODS

Details of materials and test methods are given in the notes accompanying the tables. In general duplicate sets of strength specimens were made for each method of curing for test at ages of 1 day to 1 year. One set was tested as cured and the other set before testing was soaked in water at 70°F. for 1 to 7 days, depending on the age of the specimens, in order to bring them as near as possible to same weight as when removed from the molds. Previous tests showed that the moisture content of specimens at time of test had a marked influence on strength. This, therefore, was an important feature as in many earlier tests, error was introduced because specimens cured by different methods were not of the same moisture content when tested.

Ball indentation tests were made on the tops of portions of the beams after testing in flexure in order to obtain information on the surface hardness of the concrete.

TABLE I—DATA OF COMPRESSION, WEAR AND INDENTATION TESTS

In general, each strength value is the average of 4 tests made on different days; depths of wear are the average of 5 or 10 tests; indentation loads are the averages of 9 tests, 3 each on 3 different specimens.

Values in parentheses are the ratios, expressed as percentages, of the strengths for a given method of curing and age to the strengths of water-cured specimens at the same age.

For further details of tests see notes accompanying tables.

Method of Curing	Condition When Tested	Compressive Strength lb. per sq. in. 1d.	3d.	7d.	14d.	28d.	3m.	1y.	Depth of Wear, In. 28d.	3m.	Indentation Load, Lb. 28d.	1yr.
Mold 1d. then in water at 70°F. until test	As Cured	690	1720	2770	3760	4950	5970	7230	0.47a	0.34b	9670	12620
	(Soaked)	(100)	(100)	(100)	(100)	(100)	(100)	(100)				
	Dry	1210	2120	3120	3880	4780	5510	7430d	—	0.41a	9370	9950
		(175)	(123)	(113)	(103)	(97)	(92)	(102)				
Mold 1d. then in moist room until test.	Damp	740	1860	2910	3770	4870	5830	7220	—	—	—	—
		(107)	(108)	(105)	(100)	(98)	(98)	(100)				
Mold 1d.; moist room 13d.	As Cured	—	—	—	4120	5350	5630	5640	0.53	0.44	12060	12230
					(110)	(108)	(94)	(78)				
	Soaked	—	—	—	4120	4590	4980	4720	—	—	10980	10750
					(110)	(93)	(83)	(65)				
Mold 1d.; moist room 6d.	As Cured	—	—	2990	4300	5140	5320	5080	0.50	0.45	11390	11880
				(108)	(114)	(104)	(88)	(70)				
	Soaked	—	—	2990	3640	4260	4130	4390	—	—	9170	13500
				(108)	(97)	(86)	(69)	(61)				
	Dry	—	—	3180	4010	4520	4450	5300d	—	—	9320	11210
				(115)	(107)	(91)	(75)	(73)				
Mold 1d.; moist room 2d.	As Cured	—	1970	3040	3920	4360	4530	4630	0.45	0.55	9770	10980
			(115)	(110)	(104)	(88)	(76)	(64)				
	Soaked	—	1970	2900	3470	3680	3830	3820	—	—	8410	9600
			(115)	(105)	(92)	(74)	(64)	(53)				
Mold 1d.; uncovered; air of Laboratory	As Cured	—	1880	2840	3260	3580	3540	3620	0.66	0.69	8340	9050
			(109)	(103)	(87)	(72)	(59)	(50)				
	Soaked	—	1820	2340	2780	2920	2830	3160	—	—	8750	9150
			(106)	(85)	(74)	(59)	(47)	(44)				
	Dry	—	2060	2650	3000	3040	3120	3620d	—	—	6590	8600
			(120)	(96	(80)	(61)	(52)	(50)				
Hunt Process	As Cured	—	1880	3180	4160	5060	5790	5830	0.55	0.40	7230	9860
			(109)	(115)	(111)	(102)	(97)	(81)				
	Soaked	—	1990	3060	4110	4880	4660	5140	—	—	8200	8190
			(116)	(110)	(109)	(99)	(78)	(71)				
	Dry	—	—	—	—	4540	4360	6000d	—	—	7560	7990
						(92)	(73)	(83)				
Hunt Process; in molds until test	Soaked	—	—	—	3320	3980	4510	4650	0.48	0.39	8710	9020
					(88)	(80)	(76)	(64)				
Tarvia K. P. Coating	As Cured	—	—	3210	—	5160	5570	6010	0.45	0.45	6010	7250
				(116)		(104)	(93)	(83)				
	Soaked	—	—	3180	—	5040	4950	5460	—	—	8500	8160
				(115)		(102)	(83)	(76)				
Curecrete	As Cured	—	—	—	—	4800	5330	5210	—	0.38	6500	8180
						(97)	(89)	(72)				
	Soaked	—	—	—	—	4220	4460	4780	—	—	7490	8270
						(85)	(75)	(66)				
Paraffin Coating	As Cured	—	1870	2860	4000	4620	5640	5300	0.43	0.42	9920	9790
			(190)	(103)	(106)	(93)	(94)	(73)				
	Soaked	—	1820	2920	3830	4380	4460	4300	0.45c	—	8690	9450
			(106)	(105)	(102)	(88)	(75)	(59)				
Linseed Oil Coating	As Cured	—	—	3130	—	4870	5410	5930	0.49	0.40	6980	7580
				(113)		(98)	(91)	(82)				
	Soaked	—	—	2760	—	4410	4560	5140	—	—	6090	5840
				(100)		(89)	(76)	(71)				
Sodium Silicate Coating	As Cured	—	1870	2870	3530	3760	3680	4030	0.73	0.65	6880	10790
			(109)	(104)	(94)	(76)	(62)	(56)				
	Soaked	—	1740	2570	3020	3240	3140	3640	—	—	7310	7140
			(101)	(93)	(80)	(65)	(53)	(50)				
2% $CaCl_2$ admixture; in mold 1d. then in water at 70°F.	As Cured (Soaked)	1220	2170	3200	3880	4440	5920	7030	0.53a	0.45a	9690	9320
		(172)	(126)	(115)	(103)	(90)	(99)	(97)				
2% $CaCl_2$ admixture in mold 1d. uncovered, then air of Lab.	As Cured	1220	2400	3240	3800	4120	4140	4240	0.58	0.60	8300	8810
		(172)	(140)	(117)	(101)	(83)	(69)	(59)				
	Soaked	1220	2220	2860	3140	3280	3260	3570	—	—	6560	6730
		(172)	(129)	(103)	(83)	(66)	(55)	(49)				

TABLE I—DATA OF COMPRESSION, WEAR AND INDENTATION TESTS (CONTINUED)

Method of Curing	Condition When Tested	Compressive Strength lb. per sq. in. 1d.	3d.	7d.	14d.	28d.	3m.	1y.	Depth of Wear, In. 28d.	3m.	Indentation Load, Lb. 28d.	1yr.
$CaCl_2$ surface covering, 2½ lb. per sq. yd.; in molds in air of Lab. until tested.	As Cured	—	—	3170	—	4850	5960	7060	—	0.50	8210	9260
				(114)		(98)	(100)	(98)				
	Soaked	—	—	3000	—	4610	5420	6120	—	—	6390	8200
				(108)		(93)	(91)	(85)				
Uncovered in molds in air of Lab. until tested.	As Cured	—	—	3160	—	4990	5990	7000	—	0.54	8280	9210
				(114)		(99)	(100)	(97)				
	Soaked	—	—	3000	—	4910	5580	6160	—	—	8790	7160
				(108)		(101)	(93)	(85)				
In Sisalkraft paper bags	As Cured	—	—	—	—	4880	6240	6110	—	0.40	10220	10880
						(99)	(104)	(85)				
	Soaked	—	—	—	—	4890	5770	5260	—	—	8250	7930
						(99)	(97)	(73)				

a Wear blocks removed from water at age of 14 days and stored in air of Laboratory until test.
b Wear blocks cured in water at 70°F. for 2½ months then in air until test.
c Paraffin not removed before testing wear blocks.
d 1-yr. specimens dried in air blast for 2 weeks prior to testing.

Wear tests were made in a Talbot-Jones Rattler at ages of 28 days and 3 months. Upon completion of the wear tests a few wear blocks were dried to constant weight and placed in old crank case oil for 3 or 10 days and then broken and the depth of penetration of oil noted. This treatment afforded some indication of the relative porosities of the concretes cured by the different methods. Other wear blocks were cured by the various methods and tested for absorption of water at age of 3 months after drying to constant weight. Parallel with strength and wear tests, measurements were made to determine the loss of moisture from beams and cylinders after various periods of curing.

DATA OF TESTS

The principal results of the tests are given in the following tables and diagrams:

Table 1—Data of Compression, Wear and Indentation Tests.
Table 2—Data of Beam Tests.
Table 3—Change in Weight of Specimens During Curing.
Figs. 1 and 1A—Compressive Strength of Concrete for Different Methods of Curing.
Figs. 2 and 2A—Modulus of Rupture of Concrete for Different Methods of Curing. (1/3 point loading).
Figs. 3 and 3A—Modulus of Rupture of Concrete for Different Methods of Curing. (Cantilever loading).
Fig. 4—Compressive and Flexural Strength, Depth of Wear and Change in Weight of Concrete Cured by Different Methods. Tests at age of 3 mo.
Fig. 5—Compressive and Flexural Strength and Change in Weight of Concrete Cured by Different Methods. Tests at age of 1 year.

TABLE II—DATA OF BEAM TESTS

Beams loaded at the 1/3 points of a 36-in. span. Cantilever loading tests made on portions of beams after testing under 1/3 point loading. Overhang 12 in., anchor arm 6 in.
In general, each value is the average of 4 tests made on different days.
Values in parentheses are the ratios, expressed as percentages, of the strengths for a given method of loading, curing and age to the strengths of water-cured specimens.
For further details of tests, see notes accompanying tables.

Method of Curing	Condition When Tested	Modulus of Rupture—lb. per sq. in.													
		1/3 Point Loading							Cantilever Loading						
		1d.	3d.	7d.	14d.	28d.	3m.	1y.	1d.	3d.	7d.	14d.	28d.	3m.	1y.
Mold 1d. then in water at 70° F. until test.	As Cured	165	345	490	585	685	825	865	240	460	630	725	780	990	1000
	(Soaked)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)	(100)
	Dry	125	290	355	385	450	360	525*	150	395	515	525	630	590	640*
		(76)	(84)	(72)	(66)	(66)	(44)	(61)	(63)	(86)	(82)	(72)	(81)	(60)	(64)
Mold 1d.; moist room 13d.	As Cured	—	—	—	575	505	635	735	—	—	—	695	650	850	870
					(98)	(74)	(77)	(85)				(96)	(83)	(86)	(87)
	Soaked	—	—	—	575	685	890	710	—	—	—	695	840	1070	805
					(98)	(100)	(108)	(82)				(96)	(108)	(108)	(80)
Mold 1d.; moist room 6d.	As Cured	—	—	490	460	520	615	740	—	—	580	600	690	835	900
				(100)	(79)	(76)	(75)	(86)			(92)	(83)	(88)	(84)	(90)
	Soaked	—	—	490	630	650	800	615	—	—	580	775	890	950	690
				(100)	(108)	(95)	(97)	(71)			(92)	(107)	(114)	(96)	(69)
	Dry	—	—	330	395	440	375	670*	—	—	485	470	575	565	790*
				(67)	(68)	(64)	(85)	(78)			(77)	(65)	(74)	(57)	(79)
Mold 1d; moist room 2d.	As Cured	—	360	440	445	490	605	705	—	460	550	600	635	830	800
			(104)	(90)	(76)	(72)	(73)	(82)		(100)	(87)	(83)	(81)	(84)	(80)
	Soaked	—	360	490	585	620	745	545	—	460	620	705	855	875	700
			(104)	(100)	(100)	(90)	(90)	(63)		(100)	(98)	(97)	(110)	(88)	(70)
Mold 1d.; uncovered; air of Lab.	As Cured	—	335	380	410	445	580	585	—	425	510	525	520	715	680
			(97)	(78)	(70)	(65)	(70)	(68)		(92)	(81)	(72)	(67)	(72)	(68)
	Soaked	—	360	425	485	540	590	490	—	435	525	555	675	710	555
			(104)	(87)	(83)	(79)	(71)	(57)		(95)	(83)	(77)	(86)	(72)	(55)
	Dry	—	225	320	315	375	340	580*	—	325	365	450	490	505	680*
			(65)	(65)	(54)	(55)	(41)	(67)		(71)	(79)	(71)	(68)	(51)	(68)
Hunt Process	As Cured	—	380	515	550	570	555	740	—	495	625	680	680	710	900
			(110)	(105)	(94)	(83)	(67)	(86)		(108)	(99)	(94)	(87)	(72)	(90)
	Soaked	—	380	515	645	700	870	920	—	470	630	725	875	1030	1070
			(110)	(105)	(110)	(102)	(105)	(106)		(102)	(100)	(100)	(112)	(104)	(107)
	Dry	—	—	—	—	420	375	665*	—	—	—	—	545	570	835*
						(61)	(45)	(77)					(70)	(58)	(83)
Hunt Process; in molds until test.	Soaked	—	—	—	610	640	830	800	—	—	—	770	890	1055	1020
					(104)	(93)	(101)	(93)				(106)	(114)	(107)	(102)
Tarvia K. P. Coating	As Cured	—	—	515	—	640	645	770	—	—	615	—	780	790	915
				(105)		(93)	(78)	(89)			(98)		(100)	(80)	(91)
	Soaked	—	—	515	—	720	865	995	—	—	690	—	835	1080	1150
				(105)		(105)	(105)	(115)			(110)		(107)	(109)	(115)
Curcrete	As Cured	—	—	—	—	485	565	765	—	—	—	—	710	730	915
						(71)	(69)	(88)					(91)	(74)	(91)
	Soaked	—	—	—	—	690	855	860	—	—	—	—	900	1015	990
						(101)	(104)	(99)					(115)	(103)	(99)
Paraffin Coating	As Cured	—	375	480	555	580	585	740	—	485	605	720	755	735	860
			(109)	(98)	(95)	(85)	(71)	(86)		(105)	(96)	(99)	(97)	(74)	(86)
	Soaked	—	340	475	595	680	750	915	—	450	605	645	845	980	1050
			(99)	(97)	(102)	(99)	(91)	(106)		(98)	(96)	(89)	(108)	(99)	(105)
Linseed Oil Coating	As Cured	—	—	495	—	630	650	775	—	—	625	—	730	740	855
				(101)	—	(92)	(79)	(90)			(99)		(94)	(75)	(85)
	Soaked	—	—	490	—	695	905	905	—	—	625	—	860	1090	1010
				(100)		(101)	(110)	(105)			(99)		(110)	(110)	(101)
Sodium Silicate Coating	As Cured	—	330	390	430	445	580	615	—	410	485	545	590	760	725
			(96)	(80)	(74)	(65)	(70)	(71)		(89)	(77)	(75)	(76)	(77)	(73)
	Soaked	—	365	460	520	550	600	460	—	470	560	655	735	770	570
			(106)	(94)	(89)	(81)	(73)	(53)		(102)	(89)	(90)	(94)	(78)	(57)
2% $CaCl_2$ admixture; in mold 1d.; then in water at 70°F.	As Cured (Soaked)	210	340	450	515	595	720	765	290	440	560	605	730	910	935
		(127)	(99)	(92)	(88)	(87)	(87)	(88)	(121)	(96)	(89)	(83)	(94)	(92)	(93)
2% $CaCl_2$ admixture; in mold 1d. uncovered, then air of Lab.	As Cured	210	300	345	375	485	520	560	290	430	440	555	605	715	830
		(127)	(87)	(70)	(64)	(71)	(63)	(65)	(121)	(94)	(70)	(77)	(78)	(72)	(83)
	Soaked	210	340	400	470	530	560	460	290	420	560	665	665	695	545
		(127)	(99)	(82)	(80)	(77)	(68)	(53)	(121)	(91)	(89)	(87)	(85)	(70)	(55)

TABLE II—DATA OF BEAM TESTE (CONTINUED)

Method of Curing	Condition When Tested	Modulus of Rupture—lb. per sq. in.												
		1/3 Point Loading							Cantilever Loading					
		1d.	3d.	7d.	14d.	28d.	3m.	1y.	1d.	3d.	7d.	14d.	28d.	3m.	1y.
$CaCl_2$ surface covering, 2½ lb. per sq. yd.; in molds in air of Lab. until tested.	As Cured	—	—	425	—	520	595	685	—	—	565	—	705	845	820
				(87)		(76)	(72)	(79)			(90)		(90)	(85)	(82)
	Soaked	—	—	480	—	570	740	815	—	—	605	—	810	1060	1020
				(98)		(83)	(90)	(94)			(96)		(104)	(107)	(102)
Uncovered in molds in air of Lab. until tested.	As Cured	—	—	420	—	465	555	640	—	—	525	—	660	820	755
				(86)		(68)	(67)	(74)			(83)		(85)	(83)	(76)
	Soaked	—	—	430	—	590	655	675	—	—	550	—	625	860	860
				(88)		(86)	(79)	(78)			(87)		(80)	(87)	(86)
In Sisalkraft paper bags	As Cured	—	—	—	—	580	630	745	—	—	—	—	720	790	815
						(85)	(76)	(86)					(92)	(80)	(81)
	Soaked	—	—	—	—	720	840	920	—	—	—	—	830	1050	1080
						(105)	(102)	(106)					(106)	(106)	(108)

*1-yr. specimen dried in air blast for 2 weeks prior to testing.

TABLE III—CHANGE IN WEIGHT OF SPECIMENS DURING CURING

In general each value is the average of 4 tests made on different days, although in some cases they represent the average of as many as 15 tests. Values are losses in weight except where preceeded by a plus sign which indicates a gain in weigh.

The weights of the specimens upon removal from the forms are used as the basis of the computations. The amount of mixing water in the freshly-molded specimens averaged 2.14 lb. per cylinder and 12 lb. per beam.

Method of Curing	Gain or Loss in Weight—% of Mixing Water											
	6x12-in. Cylinders						7x7x38-in. Beams					
	3d.	7d.	14d.	28d.	3m.	1y.	3d.	7d.	14d.	28d.	3m.	1y.
Mold 1d then in water at 70°F. until test.	+9.0	+9.2	+10.0	+13.4	+15.6	+21.7	+ 3.8	+7.4	+9.1	+11.1	+12.1	+14.2
Mold 1d. moist room 13d.	—	—	+5.3	14.1	25.8	35.5	—	—	+4.6	12.3	23.1	30.2
Mold 1d. moist room 6d.	—	+5.4	12.8	20.4	30.6	36.7	—	+1.9	13.3	18.4	27.3	35.8
Mold 1d. moist room 2d.	+3.7	19.2	26.1	32.2	36.8	41.5	+1.1	13.8	21.2	29.2	30.9	42.4
Mold 1d.; uncovered air of Laboratory	21.4	27.5	33.0	39.5	43.3	44.3	14.6	21.0	27.2	30.6	35.0	38.8
Hunt Process	3.8	6.3	12.9	17.4	27.6	31.9	2.4	3.3	5.5	8.6	17.2	24.3
Tarvia, K. P.	—	7.1	—	14.6	24.0	30.4	—	2.8	—	6.6	13.1	21.9
Curcrete	—	—	—	20.2	28.7	33.6	—	—	—	17.3	23.6	26.2
Paraffin Coating	3.8	6.7	12.4	15.6	25.6	32.8	4.9	5.1	9.0	12.9	16.2	22.1
Linseed Oil Coating	—	16.0	—	26.4	34.8	38.0	—	10.4	—	16.9	26.9	30.6
Sodium Silicate Coating	15.3	27.7	34.1	35.0	44.4	41.8	13.5	23.5	28.6	28.0	38.6	41.0
2% CaCl, admixture; in mold 1d. then in water at 70°F.	+7.8	+8.9	+9.2	+10.9	+19.2	+21.0	+4.9	+7.1	+7.7	+8.3	+11.7	+15.6
2% CaCl, admixture; in mold 1d.; uncovered, then air of Lab.	15.2	21.7	28.0	32.7	37.4	38.0	10.2	15.0	19.8	22.9	26.9	31.2
In Sisalkraft paper bags	—	—	—	5.8	15.4	27.2	—	—	—	7.6	15.0	25.1
Mold 1d. then in water at 70°F. Dried at about 220° F. over hot plate.	58.2	42.1	39.3	36.8	20.3	—	48.8	35.1	30.4	27.4	29.0	4.0a
Mold 1d. moist room 6d. then dried at 220° F. over hot plate.	—	36.5	45.0	41.7	56.3	36.3a	—	35.3	39.2	38.6	45.2	34.8a
Mold 1d.; uncovered in air of Lab. Dried at 220° F. over hot plate	56.5	60.3	56.5	64.5	61.2	46.2a	43.9	51.0	42.8	57.7	52.0	58.2a

a Dried in air blast from electric fan for 14 days prior to test.

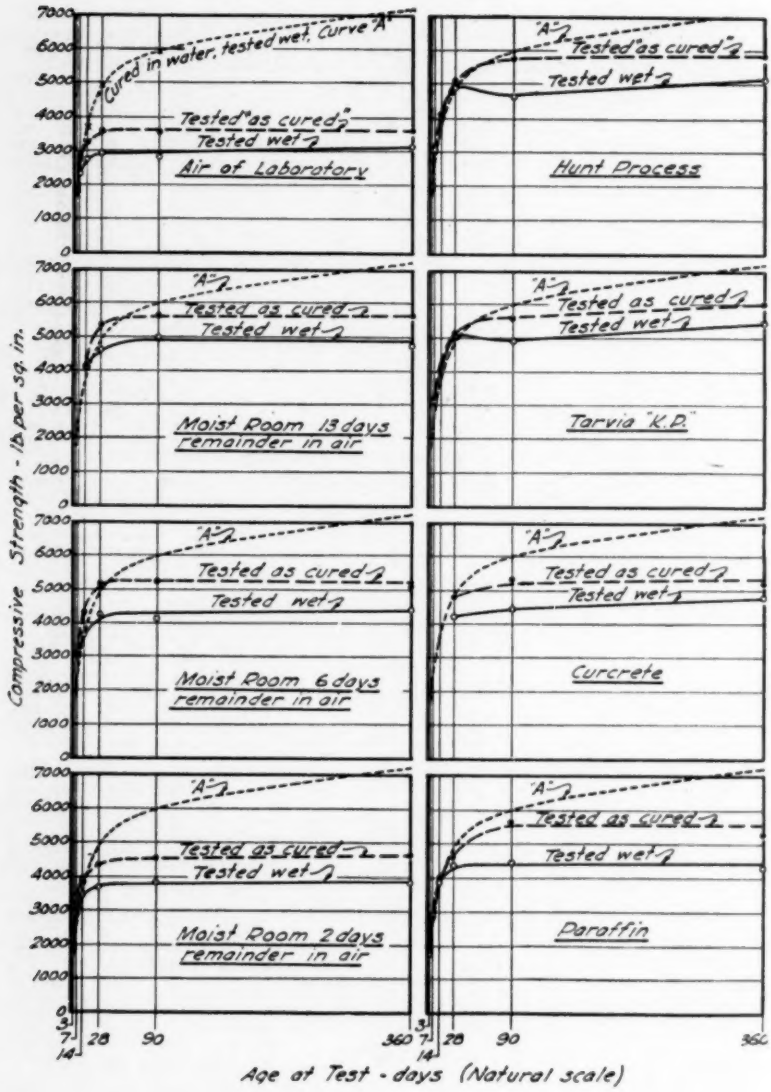
Cured in water, tested wet, Curve "A"
Tested as cured
Tested wet
Air of Laboratory
Hunt Process
Moist Room 13 days remainder in air
Tarvia "K.P."
Moist Room 6 days remainder in air
Curcrete
Moist Room 2 days remainder in air
Paraffin
Compressive Strength - lb. per sq. in.
Age at Test - days (Natural scale)

FIG. 1—COMPRESSIVE STRENGTH OF CONCRETE FOR DIFFERENT METHODS OF CURING.

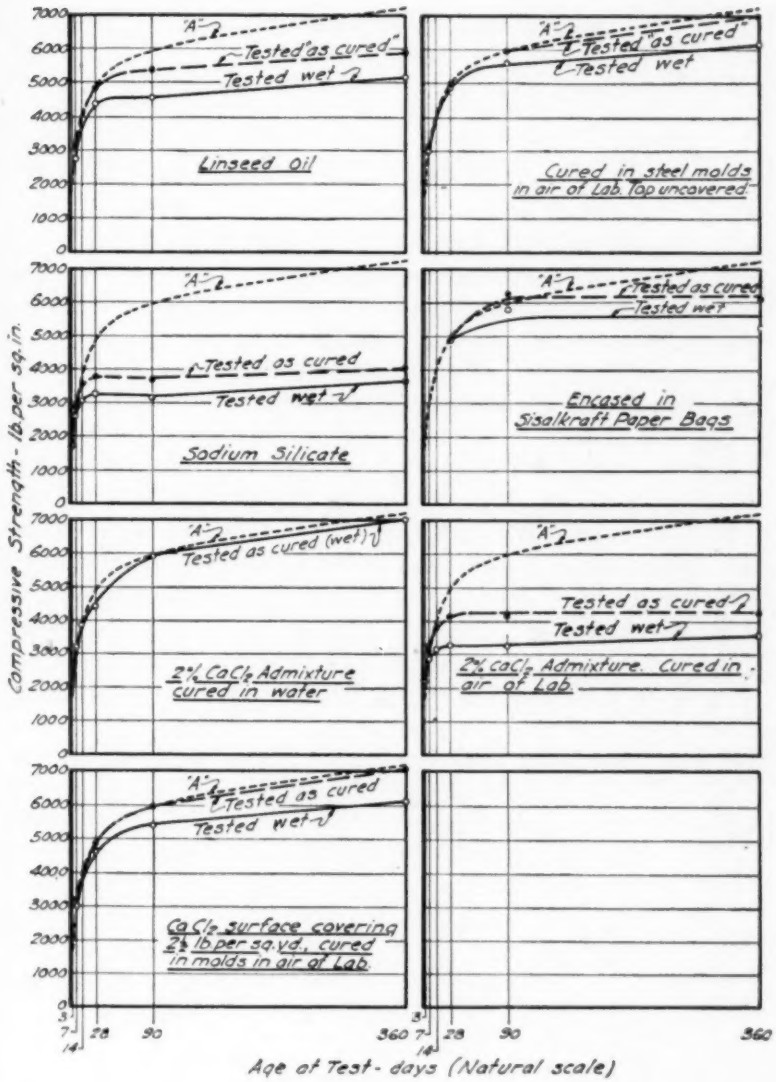
Compressive Strength - lb. per sq. in.
"A"
Tested as cured
Tested wet
Linseed Oil
Cured in steel molds in air of Lab. Top uncovered
Sodium Silicate
Encased in Sisalkraft Paper Bags
Tested as cured (wet)
2% CaCl2 Admixture cured in water
2% CaCl2 Admixture. Cured in air of Lab.
CaCl2 surface covering 2 lb. per sq. yd., cured in molds in air of Lab.
Age of Test - days (Natural scale)

FIG. 1A—COMPRESSIVE STRENGTH OF CONCRETE FOR DIFFERENT METHODS OF CURING.

Fig. 6—Relative Compressive Strengths of Concrete Cylinders Cured under Different Conditions.

Fig. 7—Relative Flexural Strengths of Concrete Beams Cured under Different Conditions. (1/3 point loading, tested wet).

Fig. 8—Change in Weight of Concrete Specimens During Curing.

Fig. 9—Comparisons of Changes in Weight of Concrete Specimens During Curing.

Fig. 10—Absorption of Concrete Blocks Cured by Different Methods. Age at test 3 months.

Fig. 11—Water Absorbed by 1-year old Concrete Specimens Cured by Different Methods.

Tables—Table 1 gives results of compression tests of 6 by 12-in. concrete cylinders, wear tests on 8 by 8 by 5-in. blocks and indentation tests (at 28 days and 1 yr.) on the top surfaces of portions of the beams after testing under 1/3 point loading. In general, compressive strengths are reported for ages of test of 1 day to 1 year for identical sets of specimens tested under the following conditions as regards moisture content: (1) In the condition that existed upon their removal from the curing room, and (2) after soaking in water for the following periods: 24 hr. for specimens tested at ages up to 28 days; 48 hr. for specimens tested at age of 3 months and from 4 to 7 days for specimens tested at age of 1 year. For a few methods of curing an attempt was made to dry the specimens before testing at ages up to 3 months by placing them over a hot plate at a temperature of about 220°F. for 24 hr. These specimens were allowed to cool to room temperature before testing. Since drying at this temperature appeared to have an injurious effect on the concrete, the 1-yr. specimens were dried at room temperature in the blast from an electric fan for two weeks prior to testing.

For purpose of comparison, the cylinder strengths in Table 1 for the different methods of curing are expressed as percentages of the strengths of water-cured cylinders tested wet at corresponding ages. The strengths of the specimens cured in water at 70°F. were used as the basis of comparison since this method may be considered as an ideal method of curing.

The depths of wear in Table 1 are for tests on blocks in an air-dry condition at ages of 28 days and 3 months.

The loads in the ball penetration tests reported for ages of 28 days and 1 year give some indication of the type of results secured by this method of testing. It was difficult to obtain

concordant results in the ball penetration tests as the loads required to embed the ½-in. steel ball to one-half its diameter were influenced to a considerable extent by the presence of coarse aggregate particles. Penetration tests were made at the other ages, but the results were not considered of sufficient importance to justify their inclusion in the table.

Table 2 gives the modulus of rupture of 7 by 7 by 38-in. beams cured by the various methods and loaded at ages of 1 day to 1 year at the 1/3 points of a 36-in. span. After loading by this method a portion of the tested beam was loaded as a cantilever using an overhang of 12 in. and an anchorage arm of 6 in. The moduli of rupture for the different methods of curing and conditions of test may be compared with those for water-cured beams tested wet at corresponding ages by means of the strength ratios given in parentheses in the table.

Table 3 gives data on the change in weight of specimens during curing. Since the change in weight was due to the specimens taking up or losing water, the change in weight at any period is expressed as a percentage of the amount of mixing water in the specimens at the time of molding.

Figures—Figs. 1 and 1A show compressive strengths of concrete cylinders at ages up to 1 year, for each method of curing. Three curves are shown on each diagram. The "A" curve, used as the basis of comparison of the different methods of curing, is for concrete cured in water at 70°F. until time of test and tested wet. The other two curves show the strengths of cylinders for the given method of curing tested after soaking in water or in the condition that existed upon removal from the curing room.

Figs. 2, 2A, 3 and 3A are plotted in a similar manner as Fig. 1 and 1A and show the moduli of rupture of concrete beams tested under 1/3 point and under cantilever loading for each method of curing.

Figs. 4 and 5 give comparisons of results obtained with the different methods at the 3-month period, the various methods being arranged in order of descending compressive strengths of cylinders tested wet at age of 3 mo. In addition to the compressive strengths, these diagrams give the moduli of rupture for both cantilever and 1/3 point loading, depth of wear (Fig. 4 only) and

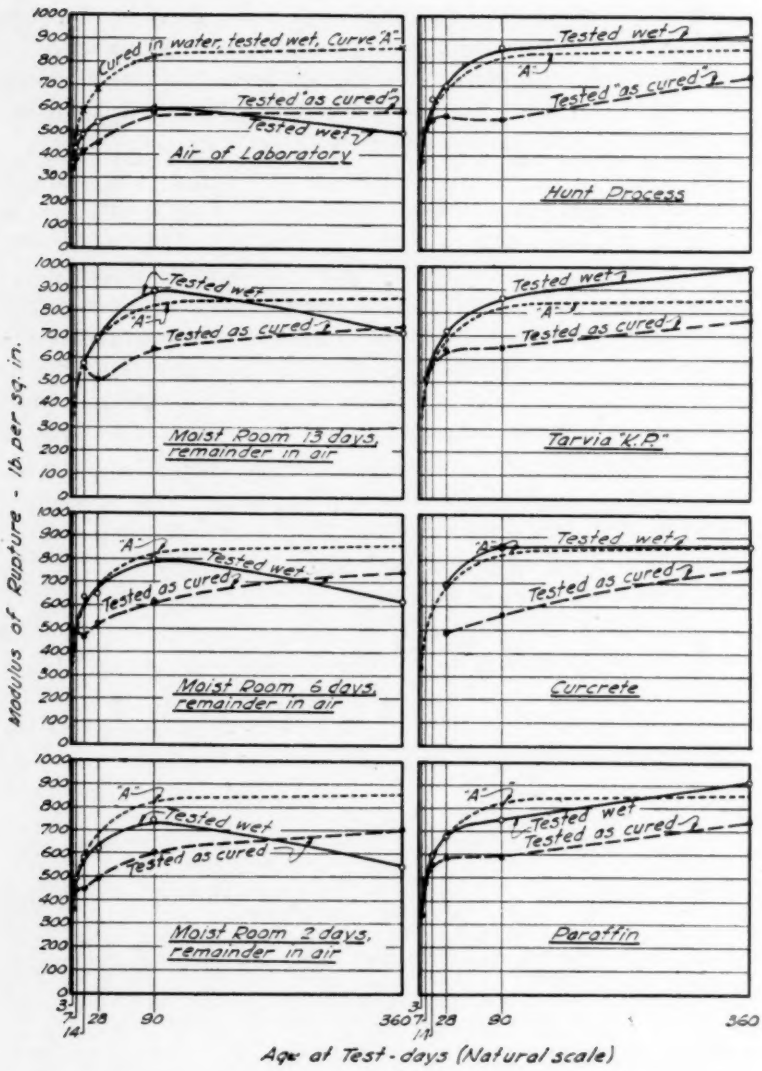
Modulus of Rupture - lb. per sq. in.
Cured in water, tested wet, Curve "A"
Tested as cured
Tested wet
Air of Laboratory
Hunt Process
Moist Room 13 days, remainder in air
Tarvia "K.P."
Moist Room 6 days, remainder in air
Curcrete
Moist Room 2 days, remainder in air
Paraffin
Age at Test - days (Natural scale)

FIG. 2—MODULUS OF RUPTURE OF CONCRETE FOR DIFFERENT METHODS OF CURING—1/3 POINT LOADING

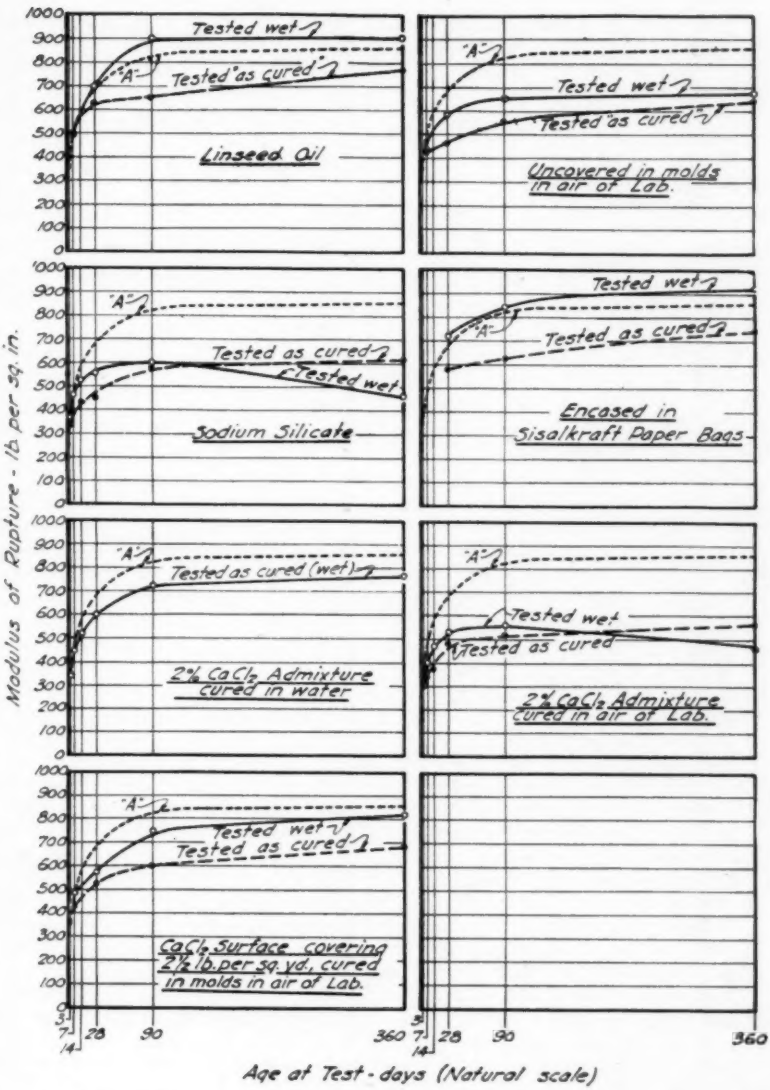
Modulus of Rupture - lb. per sq. in.
Tested wet
"A"
Tested as cured
Linseed Oil
Uncovered in molds in air of Lab.
Sodium Silicate
Encased in Sisalkraft Paper Bags
Tested as cured (wet)
2% CaCl2 Admixture cured in water
2% CaCl2 Admixture cured in air of Lab.
CaCl2 Surface covering 2½ lb. per sq. yd. cured in molds in air of Lab.
3
7
14
28
90
360
Age at Test - days (Natural scale)

FIG. 2A—MODULUS OF RUPTURE OF CONCRETE FOR DIFFERENT METHODS OF CURING—1/3 POINT LOADING.

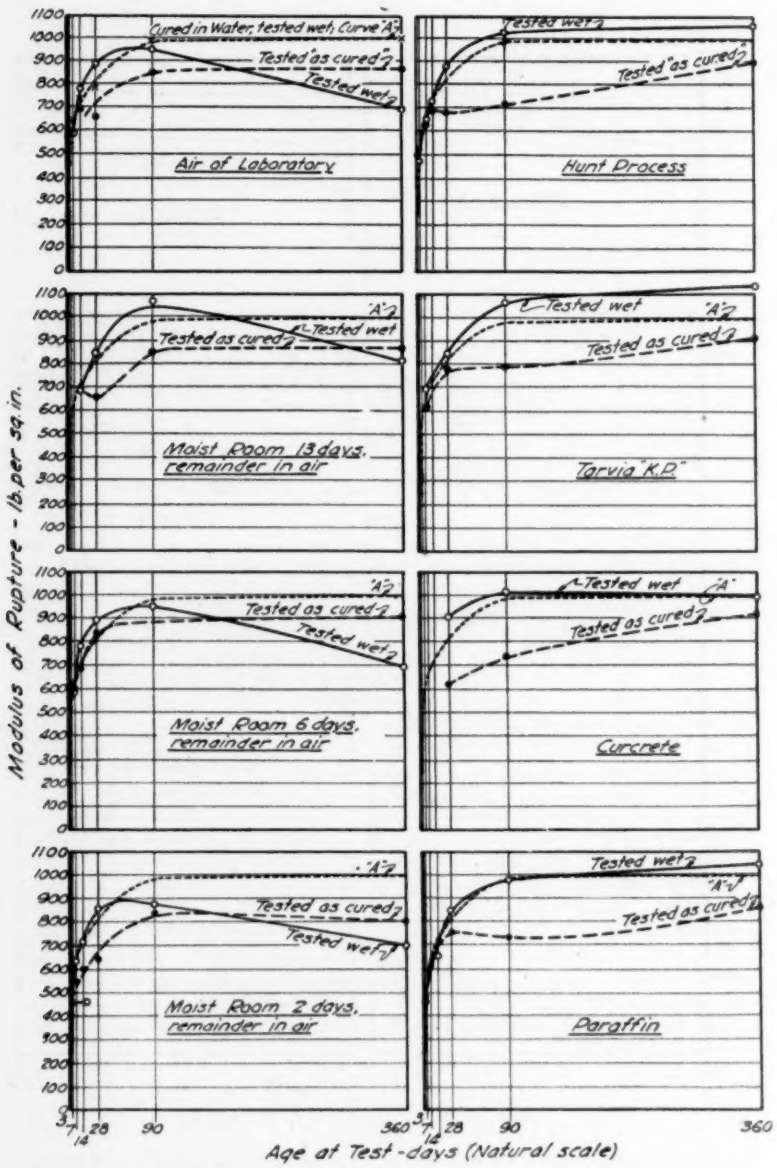
Modulus of Rupture - lb. per sq. in.
Cured in Water, tested wet, Curve "A"
Tested as cured
Tested wet
Air of Laboratory
Hunt Process
"A"
Tested wet
Tested as cured
Moist Room 13 days, remainder in air
Tarvia "K.D."
Moist Room 6 days, remainder in air
Concrete
Moist Room 2 days, remainder in air
Paraffin
3 7 14 28 90 360
Age at Test - days (Natural scale)

FIG. 3—MODULUS OF RUPTURE OF CONCRETE FOR DIFFERENT METHODS OF CURING—CANTILEVER LOADING.

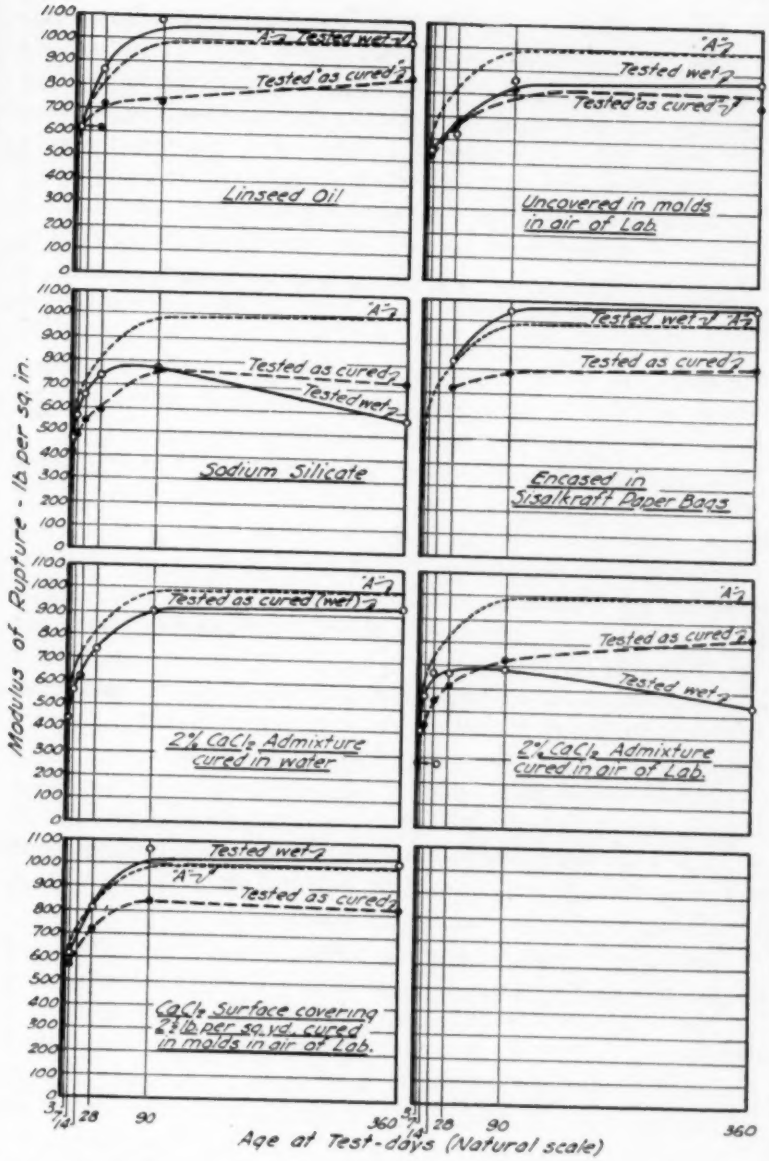
Modulus of Rupture - lb per sq. in.
Linseed Oil
Uncovered in molds in air of Lab.
Sodium Silicate
Encased in Sisalkraft Paper Bags
2% CaCl2 Admixture cured in water
2% CaCl2 Admixture cured in air of Lab.
CaCl2 Surface covering 2½ lb. per sq. yd. cured in molds in air of Lab.
Tested wet
Tested as cured
"A"
3 7/14 28 90 360
Age at Test-days (Natural scale)

FIG. 3A—MODULUS OF RUPTURE OF CONCRETE FOR DIFFERENT METHODS OF CURING—CANTILEVER LOADING.

the gain or loss in weight up to time of test expressed as a percentage of the total mixing water.

Fig. 6 shows compressive strengths for various methods at different ages compared with the strength of water-cured cylinders tested wet. The upper two diagrams in this figure are the important ones as they show relative strengths of cylinders tested in a wet condition. The two lower diagrams, which are included merely for purpose of comparison, show the strengths of cylinders tested "as cured" (in an air-dry condition) relative to those of water-cured cylinders tested wet. Thus in these two diagrams the strengths of cylinders tested in a room-dry condition on the one hand are compared with the strengths of similarly-cured cylinders tested in a wet condition on the other so that in this case the comparisons are not made on the same basis.

Fig. 7 shows the relative flexural strengths of beams for 1/3 point loading plotted in a similar manner as the curves in the upper two diagrams of Fig. 6.

The curves in Fig. 8 show the gain or loss in weight of specimens for the period between removal from forms and time of test expressed as a percentage of the total mixing water. In Fig. 8 curves for beams and cylinders are shown together for each method of curing while in Fig. 9 the curves for beams and cylinders are shown separately in order to permit easier comparison of the results for the different curing methods.

The absorption of 8 by 8 by 5-in. concrete blocks cured by different methods and tested at age of 3 months is shown in Fig. 10. In this figure the absorption shown after immersion in water for periods from 1 to 24 hr. is based on the dry weight of the block previous to soaking in water.

The curves in Fig. 11 show the rate at which the cylinders and beams absorbed water and regained or exceeded their weight at the time they were removed from the molds. These data were obtained in order to determine whether the specimens were thoroughly saturated when tested.

DISCUSSION OF TESTS

Basis of Evaluating Curing Methods from Results of Strength Tests—The proper basis on which to evaluate the effectiveness of various methods of curing from the results of strength tests is to

compare the strengths of specimens tested at the same temperature and in the same condition as regards moisture content. Thus comparisons may be based either on the results of tests on thoroughly dried or thoroughly saturated specimens. In this investigation it was considered that saturation of the specimens could be most easily accomplished and therefore the tests were planned with a view to making comparisons on the basis of saturated specimens.

In comparing the effectiveness of the various curing methods it is believed that most weight should be given to the results of the compression tests for two reasons—first, because the compression cylinders were in general more thoroughly saturated than the beams after soaking in water for the periods used, and second, the distribution and intensity of compressive strength over the cross section of the cylinder was uniform whereas the flexural tensile stress on the tension side of the beam varied in intensity from the outer fiber to the neutral axis and hence would more likely be influenced by lack of thorough soaking of the beam throughout its entire cross section than would be the case of the compression cylinder where the entire cross section is under uniform stress. In view of these considerations the relative merits of the various methods of curing so far as the strength tests are concerned are judged principally by the compressive strengths of cylinders tested in a saturated condition.

Compressive Strength of Cylinders Tested Dry and Saturated—The curves in Fig. 1 and 1A show for each method of curing the change in strength with age of cylinders tested as they came from the curing rooms and after soaking in water for various periods. These age-strength curves are quite consistent and bring out in a striking manner the differences in strength between similar specimens cured in the same manner but tested wet on the one hand and "as cured," that is, in a dry or partially dry condition on the other. The soaked cylinders always showed lower strengths than those tested "as cured" except in a few cases at early ages when about the same strengths were obtained for both the soaked and the partially dry specimens. In the case of cylinders cured with surface coatings the differences in strength between specimens tested after soaking and as cured generally became more pronounced after about the 28-day period while

with many of the other methods of curing these differences became appreciable at earlier ages. For certain of the methods, specimens tested wet had only 75 or 80 per cent of the strength of similarly cured specimens which were tested "as cured."

These marked differences in the strength of similarly cured specimens tested wet and dry or semi-dry show the necessity of basing comparisons of the strengths obtained by different methods of curing on specimens tested in either a saturated or a thoroughly dry condition.

Compressive Strength of Concrete Cured by Different Methods—Due to the difficulty of drying specimens to a constant weight rapidly, the tests in this investigation, as stated above, were planned with the view to basing comparisons of the various curing treatments on the strength of specimens soaked in water at 70°F. prior to testing, the soaking period varying with the age at test but being of sufficient duration to insure that the cylinders absorbed enough water to bring them to same or greater weight than when removed from the molds. On this basis it will be seen from Table 1 and the curves in Fig. 1 and 1A that cylinders cured by the various methods showed only minor differences in strength at ages of 1 to 7 days, most of them having strengths equal to or greater than the water-cured cylinders at these ages. The two exceptions to this were air and sodium silicate curing which gave strengths 85 and 93 per cent respectively, of that obtained for water curing at 7 days. At later ages the differences in strength for the various methods became more marked, and at 3 months and 1 year none gave strengths equal to that of water curing. This may be seen from Figures 4 and 5 where the various methods (except curing in the molds until tested) are arranged in the order of descending compressive strength of concrete at 3 months. A study of the actual strengths in these two diagrams and of the corresponding relative strengths in the upper two diagrams in Fig. 6 shows that surface coatings of Tarvia, Mc-Everlast paint (Hunt Process), linseed oil, Curcrete and paraffin and Sisalkraft bags, gave compressive strengths ranging from 75 to 85 per cent at 3 months and from 58 to 76 per cent at 1 year of those for water curing. The strengths obtained with

these particular methods were about the same as those obtained with 13 days moist room curing at all ages of test.

The lowest strengths at 3 months and 1 year were obtained with concrete containing the 2 per cent calcium chloride admixture cured in air, the sodium silicate and air-cured cylinders which were from 45 to 55 per cent of that for water curing and about 60 per cent of that for 13 days moist curing at these two ages.

Concrete containing 2 per cent calcium chloride cured in water gave markedly higher strengths than the water-cured plain concrete at ages up to 7 days but about the same strengths thereafter. Air-cured cylinders containing 2 per cent calcium chloride were likewise appreciably stronger than similar water-cured cylinders up to the 3-day period but thereafter showed a continually decreasing percentage of the strength of water-cured cylinders with age, the strength at 1 year being 49 per cent of water-cured concrete.

A significant feature of the results obtained for moist room curing for periods up to 13 days was the gradual improvement in compressive strength with increase in the duration of moist curing (Fig. 6). It is interesting to note that cylinders cured in the moist room until test gave strengths at all ages almost the same as cylinders cured in water (Table 1).

Cylinders cured in watertight steel molds with top surface uncovered or covered with an amount of flake calcium chloride equivalent to 2½ lb. per sq. yd. gave strengths closely approaching those of water-cured specimens at 3 months and about 85 per cent of the strength of water-cured cylinders at 1 year (Ref. 17 and 16, Fig. 4 and 5).

In the two lower diagrams of Fig. 6 are curves showing the relative strengths of cylinders cured by the various methods and tested "as cured"—that is, without soaking in water prior to testing. It will be noted that these curves, which compare strengths of dry or partially dry cylinders for each method with those of water-cured cylinders tested wet, give consistently higher relative strengths than their similarly cured companion specimens which were tested wet (upper two diagrams Fig. 6). This method of comparison has frequently been used in reporting results of curing tests and has led to erroneous conclusions,

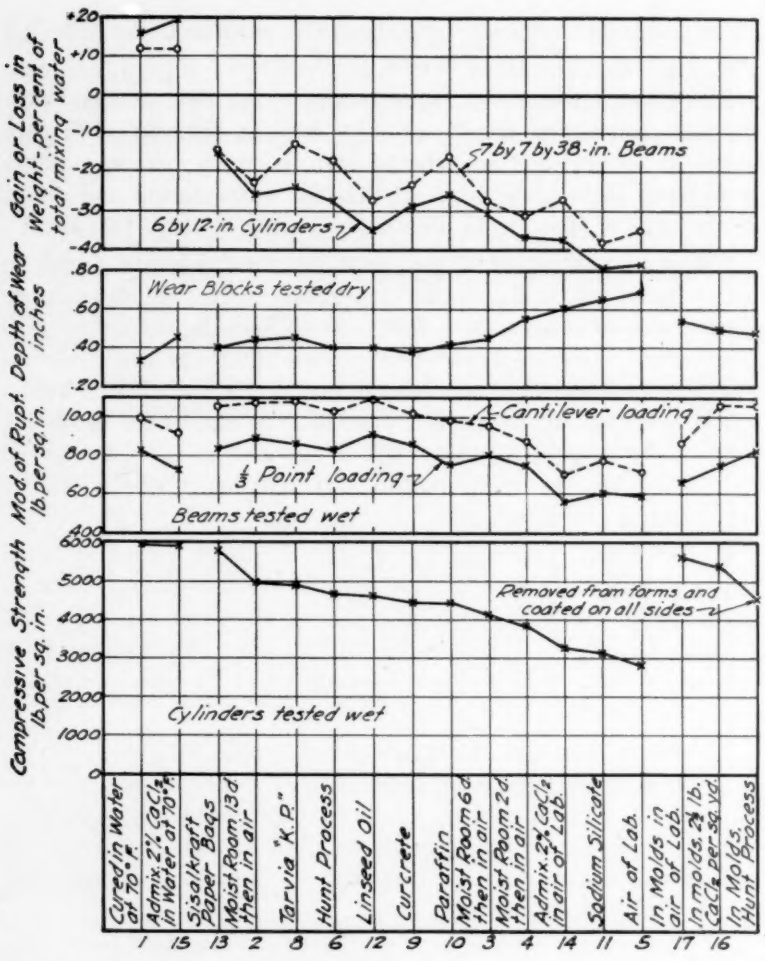

FIG. 4—COMPRESSIVE AND FLEXURAL STRENGTH, DEPTH OF WEAR, AND CHANGE IN WEIGHT OF CONCRETE CURED BY DIFFERENT METHODS. TESTS AT AGE OF 3 MONTHS. DATA ARRANGED IN ORDER OF DESCENDING COMPRESSIVE STRENGTH.

because the specimens are not of the same moisture content when tested.

Modulus of Rupture of Concrete Cured by Various Methods—The diagrams in Figs. 2 and 2A are based on data in Table 2

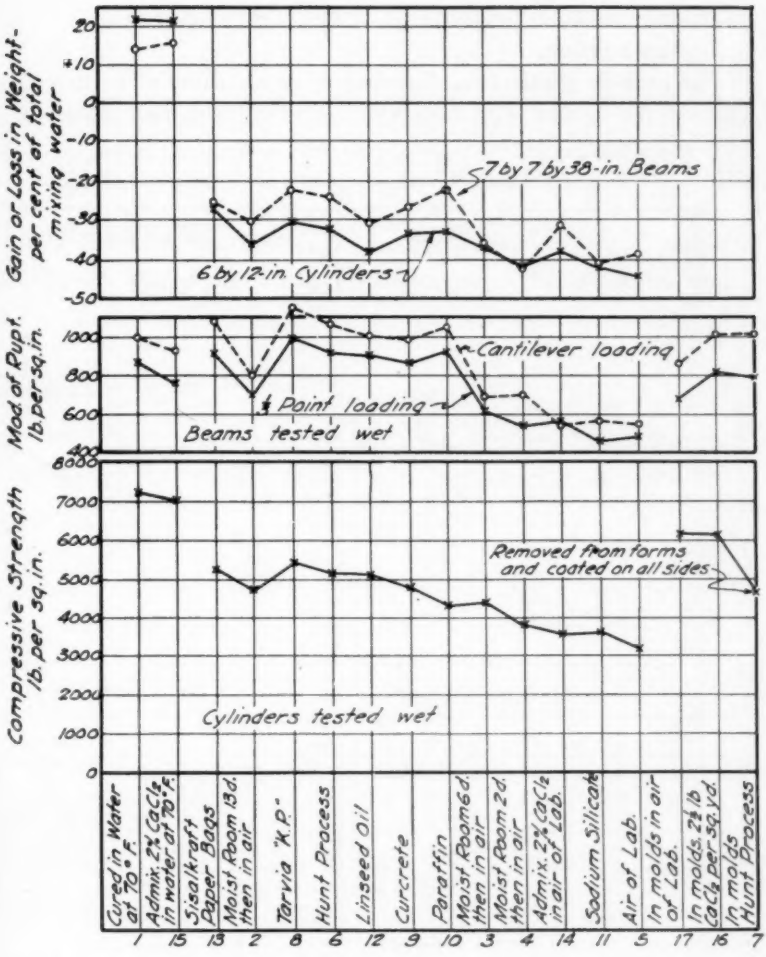

FIG. 5—COMPRESSIVE AND FLEXURAL STRENGTH AND CHANGE IN WEIGHT OF CONCRETE CURED BY DIFFERENT METHODS. TESTS AT AGE OF 1 YEAR. DATA ARRANGED IN ORDER OF DESCENDING COMPRESSIVE STRENGTH AT 3 MONTHS.

and show the relationship between the modulus of rupture of 7 by 7 by 38-in. beams tested under 1/3 point loading and age at test. Fig. 3 and 3A show similar diagrams for tests made by the

cantilever method[1] on portions of the beams after failure under 1/3 point loading.

For each method of loading values of modulus of rupture are plotted for beams cured in the same manner but tested "as

[1]See Fig. 16, page 173 *Report of Director of Research, Portland Cement Association*, November 1928.

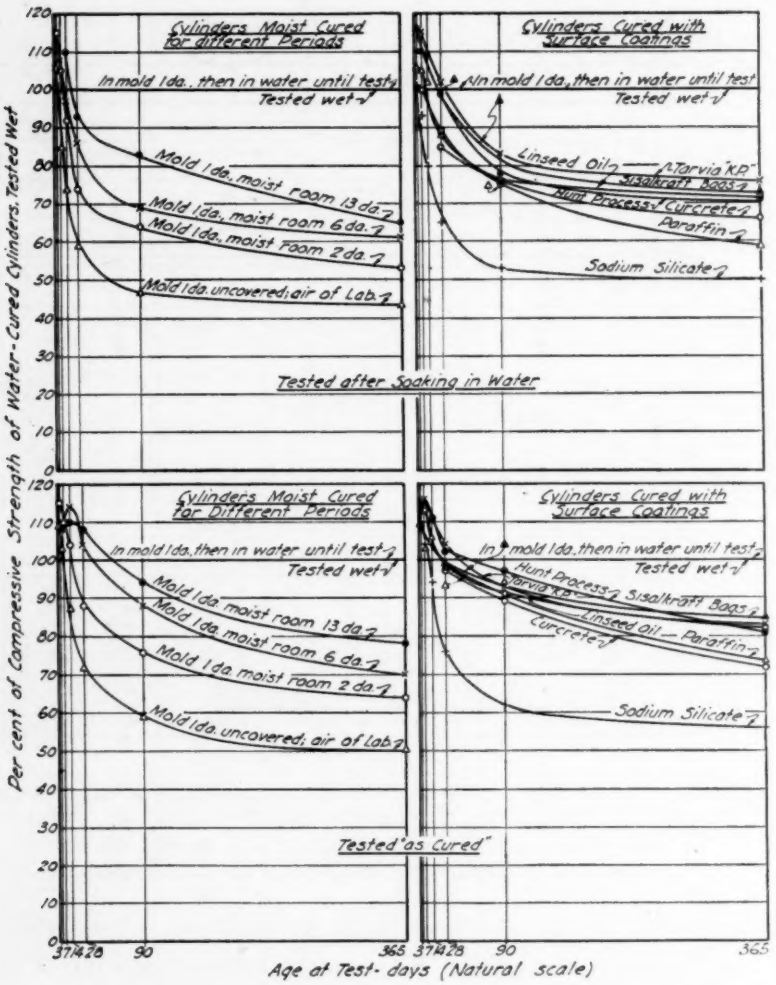
Cylinders Moist Cured for different Periods
In mold 1 da., then in water until test, Tested wet
Mold 1 da., moist room 13 da.
Mold 1 da., moist room 6 da.
Mold 1 da., moist room 2 da.
Mold 1 da. uncovered; air of Lab.
Tested after Soaking in Water
Cylinders Cured with Surface Coatings
In mold 1 da., then in water until test, Tested wet
Linseed Oil
"Tarvia KP"
Sisalkraft Bags
Hunt Process
Curcrete
Paraffin
Sodium Silicate
Cylinders Moist Cured for Different Periods
In mold 1 da., then in water until test, Tested wet
Mold 1 da., moist room 13 da.
Mold 1 da., moist room 6 da.
Mold 1 da., moist room 2 da.
Mold 1 da. uncovered; air of Lab.
Tested "as Cured"
Cylinders Cured with Surface Coatings
In mold 1 da., then in water until test, Tested wet
Hunt Process
"Tarvia KP"
Sisalkraft Bags
Linseed Oil
Paraffin
Curcrete
Sodium Silicate
Per cent of Compressive Strength of Water-Cured Cylinders, Tested Wet
0 10 20 30 40 50 60 70 80 90 100 110 120
3 7 14 28 90 365
Age at Test - days (Natural scale)

FIG. 6—RELATIVE COMPRESSIVE STRENGTHS OF CONCRETE CYLINDERS CURED UNDER DIFFERENT CONDITIONS.

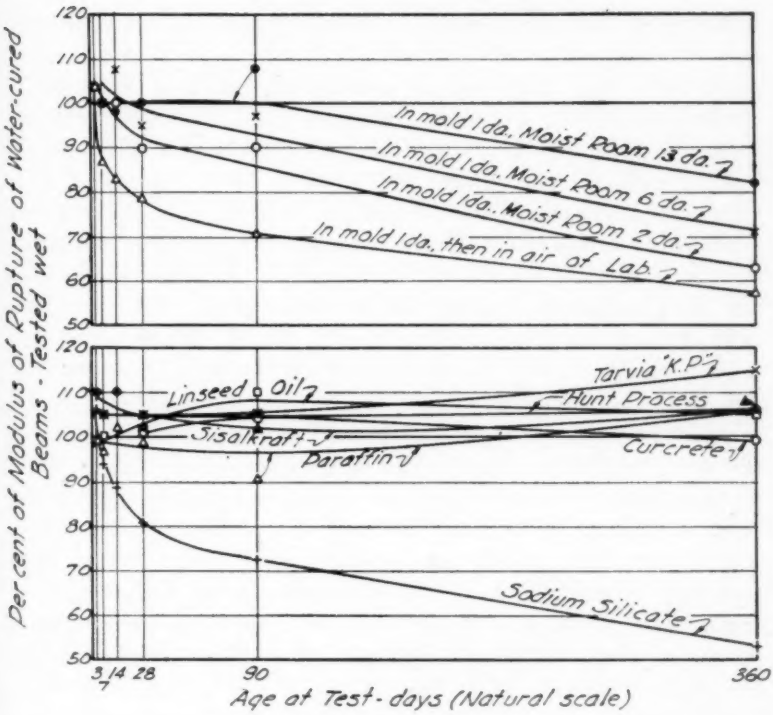

FIG. 7—RELATIVE FLEXURAL STRENGTHS OF CONCRETE BEAMS CURED UNDER DIFFERENT CONDITIONS—1/3 POINT LOADING, TESTED WET

cured" and after soaking in water prior to testing. These diagrams are of particular interest because they bring out certain marked differences in behavior of similarly cured concrete specimens when tested in flexure and compression at various ages. The following discussion of the results of the beam tests is based on the curves in Fig. 2 and 2A for 1/3 point loading; the curves in Fig. 3 and 3A for cantilever loading are similar to those in Fig. 2 and 2A differing only in that they show from 20 to 25 per cent higher values of modulus of rupture for a given method of curing and age at test.

Tests of Moist-Cured Beams—Noting beams cured for various periods in the moist room it will be seen from Fig 2 that for ages up to 3 months beams tested wet were generally stronger than

similarly cured beams tested "as cured," but at age of 1 year they showed lower strengths particularly for periods of moist room curing of 6 days and less. It is believed that this result is explained as follows: When a beam is first removed from the moist room and stored in a drying atmosphere there is a rather rapid loss of moisture from near the surfaces of the beam with a resultant shrinkage of the concrete as it dries. Since the inner portion of the beam is still moist there is little, if any, shrinkage in this portion or "core" of the beam. Under these conditions the inner moist core resists the shrinkage of the drier concrete near the surface thus inducing tensile stresses which cause the beam to fail at lower loads when tested "as cured" in the air-dry condition. When the beam is saturated prior to testing, the tensile stress near the surfaces due to shrinkage is relieved and the beam sustains a higher load than when tested "as cured." As the period of drying upon removal from moist room is extended there is greater opportunity for loss of moisture by evaporation which tends to a greater uniformity throughout the beam and therefore a reduction in the tensile stress at the surface due to shrinkage. When the beam has completely dried throughout its entire cross section, shrinkage stresses are largely, if not entirely, eliminated. If, when this condition obtains, the beam is thoroughly saturated before testing it might be expected that the strength would fall below that of a similarly cured beam tested dry, just as the strength in compression is reduced by soaking. With partial saturation in such a case, however, compressive stresses are induced in the outer fibers as only the surrounding shell of concrete is saturated at time of test.

The values of modulus of rupture obtained with the moist-cured beams at a given age increased with the duration of moist curing as may be seen from Figs. 2, 3 and 7.

Tests of Beams Cured with Surface Coatings—The moduli of rupture of beams tested wet when cured with surface coatings consisting of McEverlast paint (Hunt Process), Tarvia, Curcrete, paraffin, linseed oil and by encasing in Sisalkraft paper bags were as high or higher than those of the beams cured in water even at the one-year period. Up to the 3-month period, the results with these surface coatings parallel those obtained from beams cured in the moist room for 13 days but

differ from the latter in that they showed higher strengths at 1 year than at 3 months whereas the beams cured 13 days in moist room did not. Reference to Fig. 11 shows that the 1-year beams cured with these coatings, even when soaked in water for seven days prior to testing, were not as thoroughly saturated when tested as those cured moist for 13 days. This probably accounts for the higher strengths at one year of the beams cured with these surface coatings as compared with those moist-cured for 13 days. On the other hand, it will be noted from Fig. 11 that the 6 by 12-in. compression cylinders were in general well saturated at time of test even at the 1-year period. The better the curing the more difficult it was to thoroughly saturate specimens especially at the later ages. When surface coatings were not removed before soaking the specimens in water the absorption of water was retarded. Removal of the surface coatings used in these tests was difficult to accomplish without introducing other variables and hence was not generally attempted on the strength specimens.

It is of interest to note that when the beams were tested "as cured" the values of modulus of rupture for 13 days moist curing were about the same as for the above-mentioned surface coatings.

Beams cured with a surface coating of sodium silicate gave relatively low values of modulus of rupture which were essentially the same at all ages as those of beams cured in air of laboratory upon removal from molds. These results confirm those obtained in the compression tests for these two curing conditions and show that the coating of sodium silicate was of little value as a means of curing concrete.

Beam Tests with Calcium Chloride as an Admixture and as a Surface Covering—The moduli of rupture of beams made from concrete containing an admixture of 2 per cent calcium chloride and cured in water were 27 per cent higher at 1 day than that of water-cured beams without admixture but after the 3-day period the beams with the admixture had about 10 per cent less strength.

Air-cured beams containing 2 per cent calcium chloride showed little change in strength after 28 days and at 1 year had only 53 per cent of the strength of water-cured plain concrete beams. The strengths obtained with air-cured concrete containing the

FIG. 8—CHANGE IN WEIGHT OF CONCRETE SPECIMENS DURING CURING.

admixture were comparable to those for air-cured plain concrete at ages of 3 days to 1 year. These results indicate that although the admixture of calcium chloride accelerates the hardening of the concrete and increases the early strengths it is not a satisfactory substitute for a few days of moist curing.

A few tests were made on beams cured in wood molds until test with the top surfaces uncovered and covered with 2½ lb. calcium chloride per sq. yd. or with a coating of McEverlast paint (See Table 2 and Fig. 2A). The uncovered beams in this

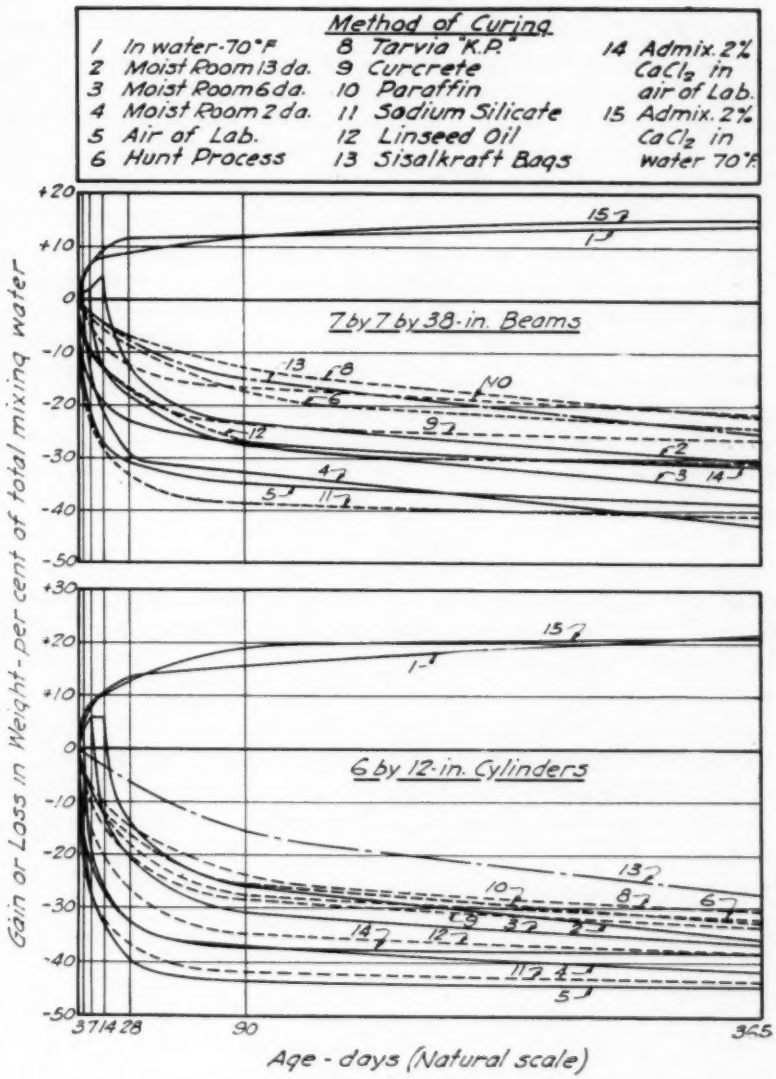

FIG. 9—COMPARISON OF CHANGES IN WEIGHT OF CONCRETE SPECIMENS DURING CURING

group of tests had flexular strengths about 80 per cent of those of water-cured beams at 3 months and 1 year while the beams covered with calcium chloride and with McEverlast paint showed

about 92 per cent and 95 per cent of the strength of the water-cured beams at these two ages. These comparisons are on the basis of specimens tested wet under 1/3 point loading.

Wear Tests of Concrete—Results of wear tests at 28 days and 3 months on 8 by 8 by 5-in. concrete blocks cured under various conditions are given in Table 1 and Fig. 4. The surface coatings on the wear blocks were not removed before test as removal of the paraffin coating (the thickest of the coatings used) in tests at 28 days was found to make little difference in the results. All wear blocks were in an air-dry condition when tested.

On the whole the results of the wear tests confirmed those of the cylinder and beam tests. Concrete cured by the methods which produced high compressive and flexural strength was most resistant to wear. This is strikingly brought out by the curves in Fig. 4 which show a gradual increase in depth of wear as compressive strength and modulus of rupture decrease.

The depths of wear of blocks tested at the 3-month period ranged from 0.34 in. for those cured in water for 2½ months to 0.69 in. for air-cured blocks. Air-cured concrete both with and without admixture of calcium chloride, concrete cured in moist room for 2 days and concrete cured with sodium silicate had relatively low resistance to wear while concrete cured by the other methods had approximately the same resistance to wear as concrete cured moist for 13 days. The results of these wear tests emphasize the harmful effect of lack of curing or insufficient curing on the wearing resistance of concrete.

Loss of Moisture During Curing—In order to determine the amount of moisture lost by the specimens at different ages all specimens were weighed upon removal from molds, from moist room, and at time of test. From these weights the loss or gain in moisture was calculated and expressed as a percentage of the mixing water originally contained in the freshly molded concrete. The data are given in Table 3 and are shown graphically in Fig. 8 and 9 for both beams and cylinders. It will be noted that in some cases the cylinders showed somewhat greater changes in weight than the beams, probably because of their somewhat smaller thickness.

A significant feature of these diagrams is the gradual but retarded loss of water from the specimens encased in Sisalkraft

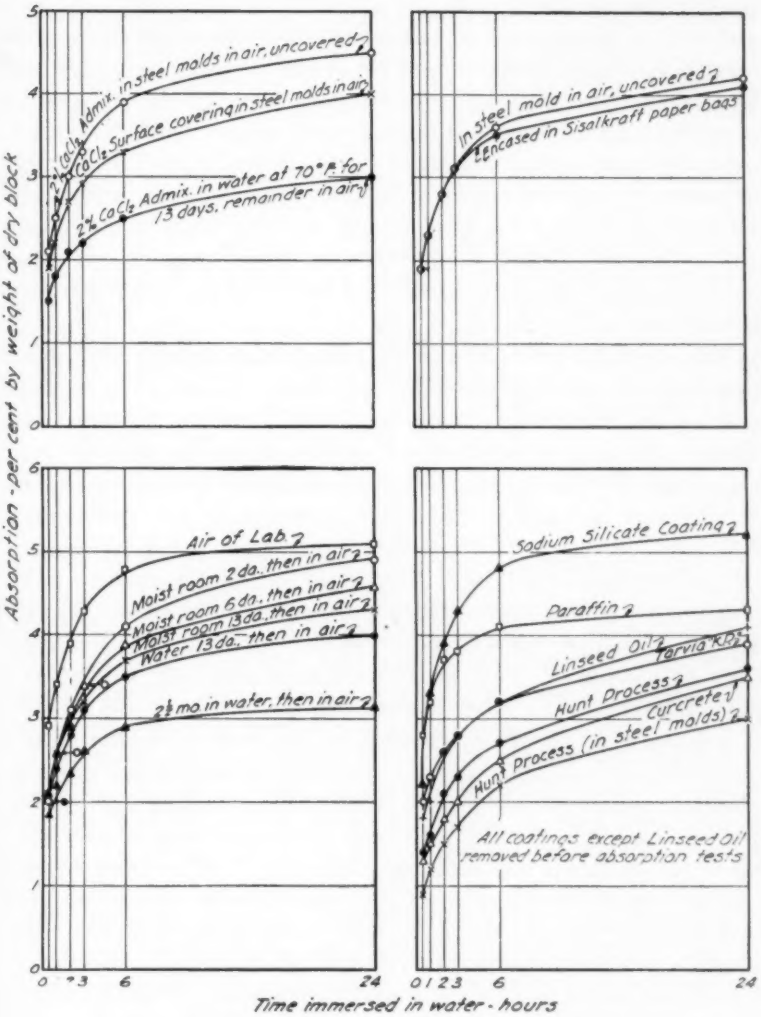
Absorption - per cent by weight of dry block
Time immersed in water - hours
2% CaCl2 Admix. in steel molds in air, uncovered
CaCl2 Surface covering in steel molds in air
2% CaCl2 Admix. in water at 70° F. for 13 days, remainder in air
In steel mold in air, uncovered
Encased in Sisalkraft paper bags
Air of Lab
Moist room 2 da., then in air
Moist room 6 da., then in air
Moist room 13 da., then in air
Water 13 da., then in air
2½ mo in water, then in air
Sodium Silicate Coating
Paraffin
Linseed Oil
Tarvia RT2
Hunt Process
Curcrete
Hunt Process (in steel molds)
All coatings except Linseed Oil removed before absorption tests

FIG. 10—ABSORPTION OF CONCRETE BLOCKS CURED BY DIFFERENT METHODS.
AGE AT TEST: 3 MONTHS;
SIZE OF BLOCKS: 8 BY 8 BY 5-IN.

paper bags or coated with linseed oil, paraffin and the bituminous coatings as compared with the more rapid loss for the air-cured and sodium silicate coated specimens. Specimens cured in moist room for 13 days showed fairly rapid loss in water upon removal from moist room while at the 1-year period the loss for this method of curing was in general not much, if any, greater than that shown by coated specimens. Specimens cured in water until test gained in weight due to absorption of water by the aggregate and to the filling of air voids. A comparison of these data with those of the strength tests indicates that in general those specimens showing greatest loss of moisture at time of test also showed the lowest strengths.

Absorption Tests of Concrete Blocks—Before making absorption tests, surface coatings were removed where possible. The paraffin coating was removed by heating, the McEverlast, Curcrete and Tarvia coatings were removed as completely as possible with kerosene while the sodium silicate was removed with warm water. No attempt was made to remove the linseed oil as it had penetrated into the concrete. The specimens were dried for 24 hours at a temperature of about 100°C. before immersion in water. The results of these absorption tests are shown by the curves in Fig. 10. They confirm the results of the strength and wear tests in that they show lowest absorptions for the methods of curing which gave highest strength and resistance to wear.

A few special absorption tests were made at the 1-year period on sections of air-cured and Tarvia-cured beams for the purpose of observing the depth of penetration of water into the concrete. The results showed that air-cured specimens were thoroughly saturated after 24 hr. soaking while the Tarvia-cured beams (coating not removed) showed little absorption even after immersion for a period of one week. The entire outer surface of the Tarvia-cured beams was then chipped off to a depth of approximately ½-in. and the process of soaking repeated. This procedure was repeated several times, one-half to one-inch of concrete being removed each time before the specimens were immersed in water. The results indicated that the inner core of concrete was as impermeable as the outer portion of the specimen and showed the difficulty of thoroughly saturating well-cured specimens of a high quality concrete.

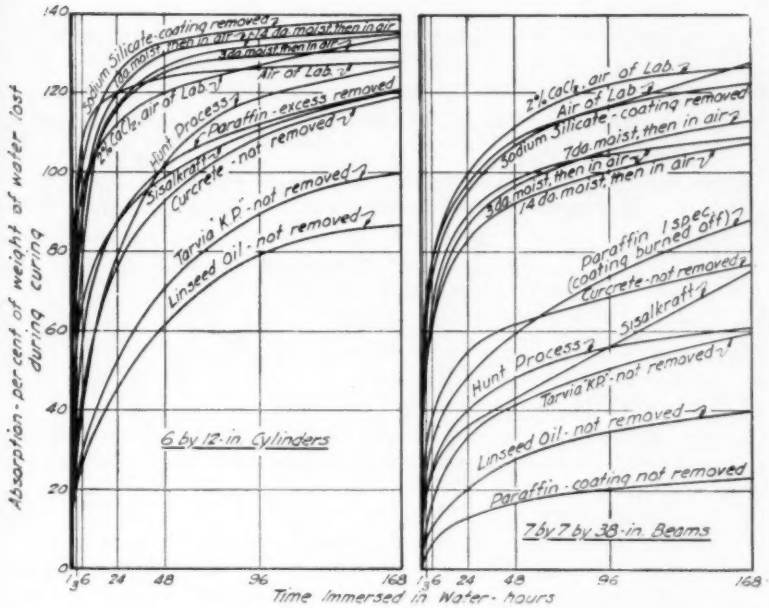
Absorption - per cent of weight of water lost during curing
Time Immersed in Water - hours
Sodium Silicate - coating removed
1 da. moist, then in air
3 da. moist, then in air
7 da. moist, then in air
14 da. moist, then in air
Air of Lab.
2% CaCl2, Air of Lab.
Hunt Process
Paraffin - excess removed
Sisalkraft
Concrete - not removed
Tarvia "K.P." - not removed
Linseed Oil - not removed
6 by 12-in. Cylinders
Paraffin (1 spec. coating burned off)
Concrete - not removed
Paraffin - coating not removed
7 by 7 by 38-in. Beams

FIG. 11—WATER ABSORBED BY 1-YEAR OLD CONCRETE SPECIMENS CURED BY DIFFERENT METHODS.

Indentation tests—The indentation tests were somewhat erratic and therefore cannot be relied upon as a true criterion of the surface hardness of the concrete or the relative merits of the curing methods employed. Due to the size of the aggregate used (maximum size 1½ in.) difficulty was experienced in making accurate tests, as in a number of cases the steel ball would become embedded on top of a pebble thus increasing the load considerably. While this test was not accurate enough to distinguish between curing methods giving concrete strengths somewhat alike it did show consistent differences in comparisons between the best and the poorest curing methods as shown by the strength and wear tests.

NOTES TO ACCOMPANY TABELS

Compression tests of 6 by 12-in. concrete cylinders.

Transverse tests of 7 by 7 by 38-in. concrete beams loaded on side at 1/3 points of 36-in. span. Portions of beams after failure under 1/3 point loading were loaded as a cantilever using a 12-in. overhang and 6-in. thrust.

Wear tests on 8 by 8 by 5-in. concrete blocks.

Ball indentation tests using ½-in. diameter steel ball made on top surface (as molded) of beams after transverse tests.

Mix 1-2-3 by dry and rodded volumes (5.9 sacks cement per cu. yd. concrete).

Cement: A mixture of equal parts of 4 brands of portland cement purchased in Chicago.

Aggregate: Fine; sand from Elgin, Ill., graded 0-No. 4 (F. M. = 3.00).
Coarse; gravel from Elgin, graded No. 4-1½ in. (F. M. = 7.00) combined as follows:

No. 4–⅜-in. – 25 per cent.
⅜–¾-in. – 25 per cent.
¾–1½-in. – 50 per cent.

Water: Lake Michigan water from the Chicago city supply at 70° F.; sufficient water used to give 3 to 4-in. slump; nominal water-cement ratio 6 gal. per sack; net water-cement ratio 5.4 gal. per sack.

Concrete: Mixed in batches of approximately 2 cu. ft. in a 3½ cu. ft. tilting mixer for 2 minutes after adding mixing water. One specimen of each type made from a batch unless otherwise indicated. Unless otherwise noted, specimens covered with steel plate or wet burlap while in mold; molds removed after 24 hr. and specimens cured as indicated.

6 by 12-in. cylinders molded in 3 layers in water-tight steel molds, each layer being rodded 25 times with a bullet-pointed ⅝-in. steel rod.

7 by 7 by 38-in. beams molded in oiled wood forms, concrete placed in 2 layers and each layer rodded 75 times with a ⅝-in. bullet-pointed steel rod. Top of beams finished with wood float.

8 by 8 by 5-in wear blocks molded in one layer in steel forms and rodded 25 times with a bullet-pointed ⅝-in. steel rod. Wear blocks tested in a Talbot-Jones rattler using a 200-lb. charge of 3¾-in. and 1 ⅞-in. steel balls. Rattler run 1800 revolutions at about 30 r. p. m., 900 in one direction and 900 in the other. Blocks weighed before and after test and loss in weight, calculated to average thickness of concrete removed by the test, taken as measure of wear. Wear blocks tested in air dry condition. Surface coatings on wear blocks were not removed before test as this was done in the case of the paraffin coating and found to make little difference in depth of wear.

After completion of wear tests representative blocks were dried to constant weight, immersed in old crank case oil for 3 or 10 days and then broken to determine the depth of penetration of oil.

Curing: All specimens not cured in moist room or in water at 70° F. were stored in a room, the air of which was kept as near as possible at a temperature of 70° F., and a relative humidity of 50 per cent during the entire curing period.

Specimens soaked before test were completely immersed in tank of clean water at 70° F. for 24 hr. when tested at ages up to 28 days, for 48 hr. when tested at age of 3 months and from 4 to 7 days when tested at age of 1 year.

Specimens dried before test were placed over a hot plate at a temperature of about 220° F. for about 6 hr. in the case of specimens tested at age of 1 day and for 24 hr. in tests at ages of 3 days to 3 months; the 1-yr. specimens were dried in an air blast from an electric fan in laboratory 2 weeks prior to testing.

Surface Coatings and Covering Materials: Following is a description of the surface coatings used and the method of their application:

McEverlast Paving Special used in Hunt Process was composed chiefly of Trinidad Lake asphalt and Gilsonite blended at a high temperature. One coat of the paint was sprayed on the fresh concrete with compressed air immediately after finishing at the rate of about one gallon for each 20 sq. yd. of surface. The paint did not penetrate the concrete but upon drying formed a thin coating over the surface which could be lifted with the fingers after 2 or 3 hr. while the concrete was still soft. After 24 hr. the paint had thoroughly dried and hardened forming a coating which adhered tightly to the hardened concrete.

Tarvia K. P. A cold patch tar cut back with light oil. Applied with brush when concrete was removed from forms after 24 hr. Concrete protected with wet burlap until Tarvia was applied. Single coat of the Tarvia applied cold, one gallon covering approximately 20 sq. yd. of surface. The Tarvia did not dry as rapidly as McEverlast paint but after 48 hr. had dried sufficiently so that no stickiness was observable although it was slightly soft.

Curcrete: An asphalt emulsion containing about 50 per cent water. Emulsion sprayed on fresh concrete immediately after placing, at the rate of about one gallon for each 10 sq. yd. of surface. Some difficulty was experienced in applying curcrete to vertical surfaces of specimens as it did not readily adhere to the concrete, and 2 or 3 coats had to be applied before concrete was sufficiently protected. Curcrete did not dry rapidly and at 3 mo. some stickiness was still observable.

Paraffin. One coat paraffin was applied hot with a brush forming a covering approximately $\frac{1}{16}$-in. thick. Two sets of wear blocks were cured with paraffin coating one set being tested with the coating on, and the other with coating removed. No appreciable difference in wear was found for these two sets of specimens.

Sodium silicate. Grasselli R. B. Silicate of Soda, a syrupy liquid, was mixed with sufficient water to give a density of 36 to 37° Baumé; this mixture required approximately 1 part water to 4 parts silicate by volume. Concrete protected with wet burlap until silicate was applied after 1 day in molds. Sufficient material of the required density was mixed and applied to the surface of specimens with a brush at rate of about 1 lb. silicate per sq. yd. of surface; 3 coats required for this quantity of silicate applied about an hour apart. The silicate gave a glossy varnish-like surface to the concrete.

Linseed oil. One gallon of raw linseed oil applied cold in 3 coats covering approximately 15 sq. yd. of surface. The coatings were applied at one-hour intervals. The oil discolored the concrete slightly, giving it a yellowish tinge.

Sisalkraft paper bags. One beam or six cylinders were placed in a bag made from Sisalkraft paper, the top of which was folded over several times and a weight placed upon it. The specimens appeared to be damp when removed from the bags at ages up to 28 days.

Surface covering of calcium chloride. Concrete protected with wet burlap for 24 hr. after placing when commercial flake calcium chloride containing 75 per cent of the anhydrous salt was sprinkled over top surface of specimens in forms. Deliquescence of the calcium chloride started immediately after placing but was not complete for about 24 hr. Top edges of specimens were dyked with paraffin to prevent moisture from running off. Moisture retained on top of specimens for several days before soaking into concrete or evaporating. Penetration of calcium chloride into the concrete ranged from $\frac{1}{16}$ to $\frac{3}{8}$-in.

Readers are referred to the JOURNAL *for June, 1930, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Winter Concreting Methods

Report of Committee 604

ROBERT C. JOHNSON, AUTHOR-CHAIRMAN

Mr. Johnson submits this report as the tentative, first offering from his committee to which the Institute looks for some approach to the basic material from which "recommended practice" for use in concreting at low temperatures may be written later. In detail of practice the present report is frankly concerned chiefly with building construction. Mr. Johnson's contribution was received too late by the critic members of the committee for their participation in the report as here published. He expresses the hope that their discussion and discussion by members generally will lay the foundation for a more comprehensive report for later submission to the Institute. —Editor.

INTRODUCTION

This report is prepared with the object of placing before contractors and their superintendents the why and how of winter concreting methods. Most modern construction organizations know how but very few know the why of curing methods specified. This sometimes results in fatal errors of judgment. The scope of this paper is limited to concrete construction in ordinary buildings although the principles involved are equally applicable to other types of construction less difficult to protect. Concrete is often only an incidental part of many buildings and consequently is handled by organizations that cannot specialize in this material to the exclusion of other important items of the structure.

*Immel Construction Co., Fond du Lac, Wisc.

All of the structural members of a building are designed to carry the live load or occupancy load, and the dead load or weight of the structure itself. The size of each member is determined on the basis of a maximum allowable unit compressive stress in its concrete. This stress depends upon the assumed strength of the concrete twenty-eight days after it has been placed. The object of the engineer's concrete specification is to obtain this design strength. The production of a predetermined quality of concrete requires:

1. Proper selection of materials.
2. Proper proportioning of materials.
3. Proper mixing of materials.
4. Proper placing of concrete.
5. Proper curing of concrete.

Most construction organizations rigidly control the first four items and neglect the fifth. The economies of proper selection, proportioning, mixing, and placing are well understood whereas the cost of curing has been looked upon as a total loss. Actually the cost of properly curing concrete may be more than saved in cement and form costs if the principles are understood.

CURING CONDITIONS

When concrete hardens the cement combines chemically with the water to make a paste which solidifies and binds the sand and stone together. The chemical combination is a slow process requiring approximately 28 days at 70° F. to reach the design strength. Unfortunately water evaporates into dry air at this temperature more rapidly than it combines with the cement. Because of this the surfaces of freshly placed concrete must be kept wet to prevent the escape of water from the concrete mass. If too much water is allowed to evaporate from the mass, the effect on the strength is much the same as leaving cement out of the mixture. Fig. 1 shows the effect of moist curing on the strength of concrete.

All of the specimens used in preparing Fig. 1 were standard 6-in. x 12-in. cylinders of 1:4 mix of average consistency. All cylinders were broken at an age of 120 days and those which were in damp sand the entire period test 2½ to 3 times as strong as

those exposed to room atmosphere for the same period. Protecting the concrete from excessive drying for only the first ten days shows an increase in strength of about 75 per cent. The concrete stored dry tested 2000 lbs. per sq. in. at 120 days whereas the concrete stored moist for only the first seven days and then stored dry for 113 days tested 3000 lbs. per sq. in. This is a remarkable increase in quality for so simple a procedure as moist curing. One of the most apparent effects of premature surface drying is the so-called dusting or sanding of cement finished floors. Concrete should be kept wet for seven days after it has been

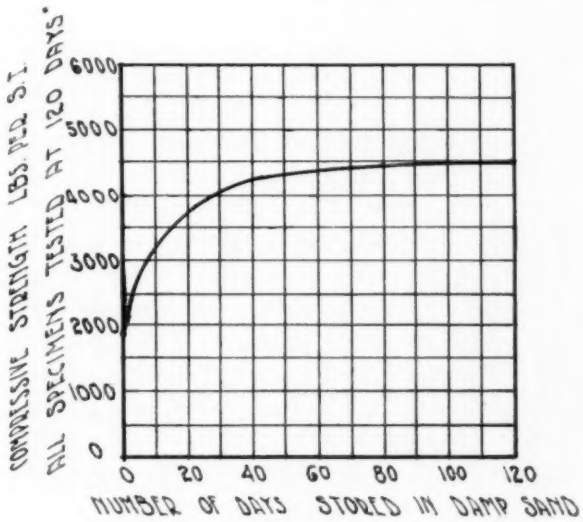

FIG. 1—EFFECT OF CURING CONDITIONS ON THE STRENGTH OF CONCRETE.

Bulletin No. 10 Structural Materials Research Laboratory, Lewis Institute.

placed. This can be done by frequent sprinkling in hot weather and by the use of escaping steam or water near salamanders in cold weather.

Temperature has a very marked effect upon the hardening of concrete. Fig. 2 shows the result of temperature upon the compressive strength of concrete.

The concrete used for all specimens in these tests was of a 1:2:4 mix with a wet consistency. A concrete designed to test 2000 lbs. at 28 days when cured at 70° F. would test only 1400 lbs. if cured at 40° F. From these curves it is apparent that concrete should be kept at a temperature of 70° F. for at least seven

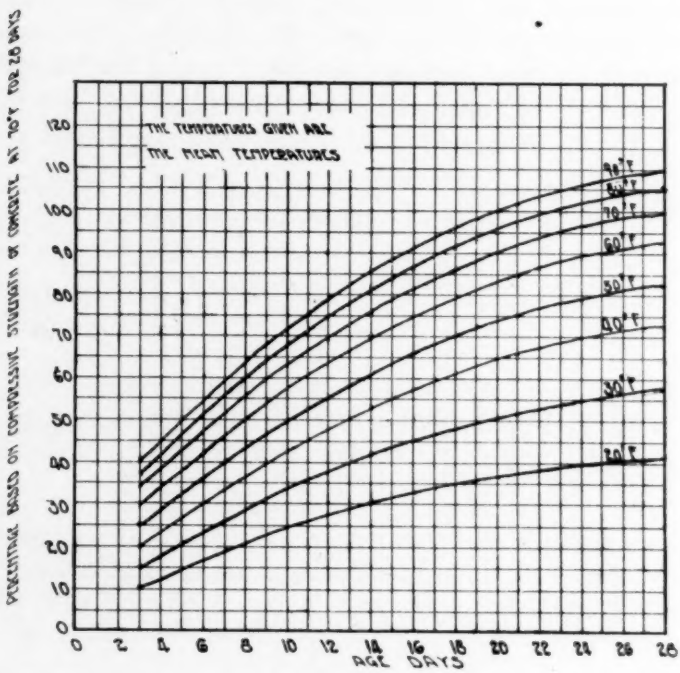

FIG. 2—PERCENTAGES OF STRENGTH FOR DIFFERENT TEMPERATURES.

Bulletin No. 81, University of Illinois.

days. The actual period should depend entirely upon the design strength, the loading, the mixture and the consistency of the concrete. Rich, dry mixes may in some instances be cheaper than an extended period of protection and heat. Table I prepared by W. A. Hoyt, M. A. S. C. E. shows the approximate daily increase in the strength of a concrete designed to acquire a strength of 2000 lbs. in 28 days when cured at low temperatures

during the early stages. This table is applicable only to concrete kept above 32° F. and no 24-hour period is to be counted during which the air has reached a temperature as low as 35° F.

TABLE I—APPROXIMATE DAILY INCREASE IN STRENGTH OF CONCRETE DESIGNED TO PRODUCE A COMPRESSIVE STRENGTH OF 2000 LB. PER SQ. IN. AT 28 DAYS

(Use proportionally higher increments for higher strength concrete)

Mean Daily Temp. Degrees F.	Age in Days									
	1	2	3	4	5	6	7	8	9	10
40	95	88	70	65	56	54	52	50	47	45
44	112	93	75	66	60	56	52	51	48	47
48	133	95	79	68	61	58	54	53	51	49
52	155	100	81	69	63	61	57	55	52	50
56	175	110	83	70	66	63	59	59	54	51
60	195	122	85	71	68	65	63	60	56	53
62	204	124	87	72	70	66	63	60	56	53
64	213	126	90	74	71	67	63	60	56	53
66	222	128	93	76	73	68	63	60	57	53
68	230	130	96	78	74	69	64	60	57	53
70	240	130	100	80	75	70	65	61	57	53

Mean Daily Temp. Deg. F.	Age in Days										
	11	12	13	14	15	16	17	18	19	20	21
40	43	40	39	37	33	30	30	27	25	23	22
44	45	43	40	38	34	32	30	27	25	23	23
48	48	45	42	41	37	34	31	28	25	24	23
52	48	46	43	42	38	35	31	30	28	26	24
56	49	46	44	42	38	35	31	30	30	27	26
60	50	46	44	42	38	35	32	30	30	29	28
62	50	46	44	42	38	35	32	31	30	29	28
64	50	46	44	41	38	36	33	31	30	29	28
66	49	46	44	41	38	36	33	32	30	29	28
68	49	46	44	41	38	36	34	32	30	29	28
70	49	46	44	41	38	36	34	32	30	29	28

Example—

It is desired to estimate from the table the approximate strength at five days of concrete which has been subjected to the mean daily temperatures given:

1st day 54° F. Increment from table 155 (Taken for 52°)
2nd day 48° F. Increment from table 95
3rd day 60° F. Increment from table 85
4th day 52° F. Increment from table 69
5th day 44° F. Increment from table 60

Total.....464 lb. per sq. in.

This is less than ¼ of 2000, or 500 lb. per sq. in. and the heating should be continued.

FREEZING OF CONCRETE

Under no circumstances should freshly poured concrete be allowed to freeze as the water becomes crystallized and is not available for combination with the cement. If wet concrete has become frozen it should be thawed out slowly and kept warm and wet for at least 20 days. Concrete must never be allowed to freeze and thaw alternately or it will be past any possible salvage and have to be removed. Concrete which has partially hardened before freezing is seriously damaged because the expansion of the water loosens all of the solids in the partially hardened mass. *Never allow green concrete to freeze.*

DESIGN LOADS AND CONSTRUCTION LOADS

One of the great dangers of winter concreting is in the erection of tall reinforced concrete buildings designed for light live loads. In buildings of this class the live loads for which the floors are designed are usually less than the construction loads which occur during the erection of the building. For this reason heavier loads are likely to be placed on the floors while the concrete is still green, than the panels were designed to carry when the concrete had cured. Where concrete is not properly cured to gain full early strength, danger is imminent in winter construction. Some winter failures are due to this cause rather than to the actual freezing of the concrete.

Assume a concrete slab designed for concrete testing 2000 lbs. per sq. in. at 28 days with the following conditions:

Live load equals 40 lbs. per sq. ft.
Dead load equals 80 lbs. per sq. ft.
Total load equals 120 lbs. per sq. ft.
Unit compressive stress equals 650 lbs. per sq. in.
Then compression due to dead load equals $\frac{80}{120} \times 650 = 434$ lbs.

With these conditions in mind assume the concrete is placed in cold weather, average 40° F. and stripped in 20 days to start brick work. Brick and mortar are stored on the slab averaging in weight 100 lbs. per sq. ft. From curve 2 the strength of the con-

crete is .65 x 2000 = 1300 lbs. The total load equals 100 plus 80 = 180 lbs. and the stress in the concrete is $\frac{180}{120}$ x 650 = 975 lbs. per sq. in. From these computations the ultimate compressive strength is only 1300 lbs. instead of 2000 lbs. and the unit compressive stress is 975 lbs. instead of 650 lbs. This is a dangerous condition and one that often exists on cold weather jobs.

METHODS

(A) *Frost Protection*—Concrete should be placed in the forms not colder than 70° F. and not hotter than 140° F. and should be maintained at not less than 70° F. for at least seven days. These temperature ranges control the amount of heat that must be furnished on the job. The heating of materials is therefore a progressive process increasing with the decrease in temperature. Starting with the first chilly days in the fall, it is sufficient to heat the water. As temperatures drop lower, usually at about 35° F., the sand and stone must be heated. In heating aggregates care must be taken to see that all frozen lumps are thawed out and broken up. All frost and ice must be removed from the forms and reinforcing steel before any concrete is placed. This can best be done by the use of steam jet. Before freezing temperatures are reached it is usually sufficient to cover freshly poured concrete with canvas and marsh hay to protect it during the night.

(B) *Winter Protection*—For concrete work carried on when day time temperatures are below freezing it is necessary to enclose the structure and furnish temporary heat. For ordinary winter jobs in the Northern States the following practices have become quite general.

1. *Plant*—The following items of plant are required, their number and size depending upon the size of the job and the temperatures:

1. Steam boiler—50 h. p. 60 lb. pressure
2. Steam hose—approx. 200 ft.
3. Steam points—1½-in. for aggregates
4. Iron pipe and fittings—1½-in.
5. Thermometers

6. Canvasses—12 oz. duck 16 ft. x 20 ft.
7. Salamanders
8. Marsh hay—not always required
9. Water barrel for mixer water
10. Water barrels for fire protection

2. *Heating Materials*—A water barrel should be mounted above the mixer and connected to the steam boiler by 1½-in. pipe, valved at the mixer, with the end terminating a few inches above the bottom of the barrel. (Sketch)
Steam should be allowed to flow to the barrel in sufficient quantity to keep the water entering the mixer at about 150° F.

For ordinary jobs the aggregate piles are heated by several pieces of steam hose to which are attached 6 foot lengths of perforated pipe drawn to points at the end and closed. These points are forced into the stock piles near location where the aggregate is being removed. It is good practice to cover all stock piles with canvas to retain the heat and to keep out snow and ice. Sand and stone should be heated to a minimum of 40° F. and preferably to between 60° and 70° F. Where storage bins are used, perforated pipe coils may be placed near the bottoms of the bins. If sand and stone are received in carload lots steam points may have to be driven into the cars before they can be unloaded. In all cases all frozen lumps must be thawed out.

A steam hose outlet must be made available at each floor level to be used in removing snow and ice from the forms and reinforcing steel. This should be done continuously just ahead of placing. The varying conditions of each job control the actual plant requirements. The general principle as stated before is to place the concrete in the forms not colder than 70° F. nor hotter than 140° F.

3. *Protection*—Before starting to concrete the columns of any story, or the floor supported by such columns, canvasses of 12 oz. duck should be hung from the spandrel-beams of the floor to be poured and lashed under the spandrel-beams of the floor below. The canvasses should be held away from the outside face of the exterior concrete between 6 inches and 24 inches to provide space for the circulation of heat. The canvas curtains must be well lapped to exclude wind and must reach well below the surface

of the floor supporting the columns to be concreted. When the entire floor is not poured in one operation, the area to be poured is shut off from the rest of the story by a curtain across the building.

Salamanders containing coke fires should be placed inside the enclosure and a temperature of 70° F. should be obtained at the underside of the forms before any concrete is placed. This will generally require one salamander per 300 sq. ft. of floor area. At least one salamander will be required at each exterior column and in extremely cold or windy weather two may be required. Heat holes about 8-in. x 12 in. should be left in the floor being concreted with one hole to approximately each salamander. These vents serve the double purpose of allowing heat to reach the top of the slab and letting the carbon monoxide gas escape. Whenever a considerable number of salamanders are being used the firing should be done by men working in pairs so that the danger of gas poisoning is minimized. In winter concreting columns are usually poured only about an hour ahead of the floor. These columns must be well hammered and tamped to limit the shrinkage cracks at the column tops.

As fast as the floor slab is concreted, it must be covered with canvas well lapped and supported 6 inches to 24 inches above the concrete. These covers must be well lapped over the side curtains and secured. The top canvasses may be supported on frames attached to the column reinforcing or on braces fastened to the vent hole boxes.

During the entire period of winter concreting a detailed record should be kept of the following temperatures:

1. Outside Air.
2. Concrete reaching forms
3. Bottom of concrete slabs
4. Top of concrete slabs
5. Bottom of exterior columns at most exposed side.

These readings should be taken every four hours, day and night, and additional heat and protection furnished if the temperature falls below 50° F. at the most exposed points. Salamanders must be fired continuously in small amounts to maintain a uniform temperature. Full water barrels and fire buckets must be available wherever salamanders are burning.

4. *Period of Protection*—The duration of the heating period as previously stated should be seven days for ordinary 2000 lb. concrete. This may be materially reduced by the use of early strength concrete or by increasing the cement content or by decreasing the water cement ratio. The detail temperature records must be studied to determine the time of protection. It is advisable to take test cylinders from the forms, storing one on the fresh concrete and one in warm, damp sand. These should be broken at seven days.

5. *Stripping of Forms*—It is impossible to give any definite rules as to the minimum time forms must remain in place on winter concrete jobs. The safe time to strip forms must be determined by the particular conditions on each structure. The temperature records and test cylinders should both be used in figuring this time.

Readers are referred to the JOURNAL *for June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930*

A Summary of the Results of Investigations Having to Do with Volumetric Changes in Cements, Mortars and Concretes, Due to Causes Other Than Stress

Report of Committee 102, Volume Changes in Concrete

BY RAYMOND E. DAVIS*, AUTHOR-CHAIRMAN

This report is the work of the author-chairman. Its scope and its detail were so formidable as to discourage critic members from going back over the ground which Professor Davis apparently has covered so thoroughly, in the time available for suggestion or discussion to be considered now.—EDITOR

INTRODUCTION

CEMENTS possess the property of changing their volume with variations in moisture conditions, just as do clays and other granular materials of similar character, and all mortars and concretes in which cement is employed are likewise affected, not only during early life when the hardening process is most actively in progress, but for all time so long as the particles of the mass are in the process of physically absorbing or giving up moisture or are undergoing chemical change. These volumetric changes due to variations other than temperature have for a long time been recognized to exist, and both in this country and abroad numerous investigators have observed and reported upon the phenomena of shrinkage accompanying the hardening of cement products in air, and of expansion accompanying the hardening of these products under water.

*Professor of Civil Engineering, University of California.

Quite independent of the volume changes just mentioned are those produced by variations in temperature which for concrete, as for other structural materials, produce expansion or contraction accordingly as the temperature rises or falls.

The purpose of this report is to bring together the results of what are considered to be some of the more important investigations so far reported, having to do with volume changes produced by causes other than stress, that is, (a) those influenced by moisture conditions and (b) those influenced by variations in the temperature of the material. Little attempt has been made to discuss the relative accuracy of published findings of the various investigations or to explain apparent discrepancies in these findings, this being left largely to the judgment of the reader.

However, with regard to moisture-volume changes, which always take place slowly and are long continued, it should be stated that the results of not a few of the researches indicate to the writer a lack of comprehension on the part of the observers as to all of the variables involved. Particularly is this true in the matter of the time factor and with reference to variations in temperature and atmospheric humidity. Thus the shrinkage of a mortar in air for three months is no criterion of the shrinkage at three years. Nor is the shrinkage of the concrete exposed to an atmosphere of, say, 70° F. and 70% relative humidity an indication of what the shrinkage would be for the same concrete exposed to a relative humidity of 50% at this same temperature. Again, it is not to be expected that a mass of thick section will exhibit the same rate of shrinkage as one of thin section; nor would the ultimate shrinkage be as great for the former as for the latter.

At the close of the report will be found a bibliography of the literature which was consulted in the preparation of the paper, each title being numbered. These reference numbers appear over the names of the authors in corresponding portions of the text of this report.

VOLUMETRIC CHANGES TAKING PLACE WITH TIME

Important factors influencing volume changes produced by the hardening process or affected by moisture conditions, which take

place with time, are discussed under the following heads:

1. Composition and Fineness of Cement.
2. Proportioning of Mix.
3. Type and Gradation of Aggregate.
4. Admixtures (Lime, Clay, etc.).
5. Consistency of Mix.
6. Moisture Conditions.
7. Age at First Observation.
8. Length of Period of Investigation.
9. Size and Shape of Mass.
10. Method of Mixing and Placing and Absorptiveness of Molds.
11. Amount and Distribution of Reinforcement.

1. *Effect of Composition and Fineness of Cement:* The volume changes among cements vary considerably with the chemical composition and are affected by fineness of grinding.

Tests reported by P. H. Bates[25] of the U. S. Bureau of Standards were made on 1 by 1 by 13-in. bars of neat cement and of 1:3 standard sand mortar, (1) using ten different brands of cement as received (75 to 91 per cent passing a No. 200 sieve), (2) using the same ten brands reground so that about 99 per cent passed a No. 200 sieve, and (3) using both the normal and reground cements after adding 2.5 per cent of SO_3 in the form of gypsum. After immersion for thirteen weeks, the average expansions of the ten bars in each group were as shown in Table 1, the initial observations having been made 24 hours after molding the bars.

TABLE 1
EFFECT OF COMPOSITION AND FINENESS OF CEMENT UPON EXPANSION (BATES)

	Expansion of Neat Cement Bars	Expansion of 1:3 Mortar Bars
Normal cement	0.076%	0.012%
Reground cement	0.085	0.013
Normal cement + 2.5% SO_3	0.154	0.028
Reground cement + 2.5% SO_3	0.121	0.017

From the table it will be seen that the expansions of the normal cements were on the average only about 90 per cent of the expansions of the reground cements for both the neat cement and the 1:3 mortar bars. It also appears that the addition of gypsum to the normal and reground cements increased the volume quite

markedly, the expansion for the normal cements being more than doubled.

These tests also demonstrated that the resulting volume changes varied considerably among the ten cements investigated, the expansion varying from 0.057 to 0.120 per cent for the normal neat specimens and from 0.005 to 0.024 per cent for the normal 1:3 mortars.

W. K. Hatt[55] at Purdue University made tests upon 2 by 2 by 24-in. bars of six brands of portland cement. The specimens were stored in water for 10 days and then in air for about 4 years. The maximum contractions, amounting to from 0.12 to 0.23 per cent were observed after about 1 year. Subsequent changes due to variations in humidity of the air were approximately alike for the several brands. The data from a second series of tests by Hatt are presented in Table 2. In this series the neat cement bars were stored in water 50 days followed by air storage for 220 days and then by water storage again for 889 days.

TABLE 2.

CHANGES IN LENGTH OF NEAT CEMENT BARS MADE FROM SEVERAL BRANDS OF CEMENT (HATT)

Storage	Actual Percentage Change During Period	Net Percentage Change at End of Period Based Upon Initial Reading
50 days in water.....	+0.01 to +0.05	+0.01 to +0.05
220 days in air.....	—0.11 to —0.16	—0.06 to —0.15
889 days in water.....	+0.18 to +0.21	+0.04 to +0.15

It will be seen that among these several cements the changes from the initial volume (net per cent changes) vary much more than do the actual percentage changes under a given condition (wet or dry). That is, the cements which exhibited the greater net expansions under water also developed the lesser net contractions in air. Thus it appears that an immersion test by itself is not a true measure of the volume changes which may be anticipated for a given cement.

This same series of tests included observations of weight changes. The results indicate that the percentage loss or gain in weight due to moisture for a given cement is indicative of (though not of necessity directly proportional to) the change in length.

Among the different cements, the ratio of the volume change to the weight change was far from being constant. That is, one cement might exhibit larger weight changes yet smaller volume changes than another cement.

Accelerated expansion tests using the Le Chatelier apparatus in which the cement paste was boiled for 6 hours were reported in *Le Ciment* (1923) as shown in Table 3.

TABLE 3

EFFECT OF BOILING CEMENT PASTE

Cement	Percentage Residue on Sieve		Expansion in mm. with Le Chatelier Apparatus
	Metric 900 U. S. Equiv. 76	Metric 4900 U. S. Equiv. 178	
A	3	28	6
	2	20	3
	1	16	1
B	2.0	25	2
	1.5	20	2
	0.5	14	½

These results indicate a pronounced decrease in the expansion with increase in fineness of the cement. This is the reverse of the findings of Bates, using bars, although one or two samples which he tested appeared to exhibit slightly less expansion with an increase in fineness of the cement. The difference in test results may perhaps be explained by the difference in test methods. It appears that the boiling test does not give results indicative of the behavior of a cement hydrated in the normal manner.

The results of tests of volume change of several cements by Dr. H. Nitzsche[35] are as shown in Table 4, where the cements are listed in order of decreasing fineness. The mortar bars (100 mm. in length and 5 sq. cm. in cross section) were of an approximate 1:4 mix and of average consistency, such as might be appropriate for reinforced concrete. They were left in the molds two days after which they were stored 7 days in water, 10 days in room air, and then dried to constant weight at 50° C. It is impossible clearly to separate the effect of the kind of cement from the effect of the degree of fineness of the cement, but it appears that at least for the portland cements the expansion in water increases with

the fineness of grinding and that in general portland cements expand more than slag cements. In any case, the results do show a distinct difference between the expansions of the cements investigated.

TABLE 4
VOLUME CHANGES IN MORTAR BARS (NITZSCHE)

Cement	Percentage Expansion in Water
Finest slag cement	0.0275
Finest portland cement	0.0347
Coarsest slag cement	0.0224
Coarsest portland cement	0.0255
Iron portland cement	0.0204

A. H. White[14] measured the volume changes of many neat cements and 1:3 cement mortars made into 1 by 1 by 4-in. bars. The range of variations of expansion and contraction for the several cements may be summarized as follows:

Neat cement bars hardening under water: After 1 year, expansion 0.07 to 0.15 per cent; on drying 65 days in air after 3 years in water, total contraction 0.13 to 0.15 per cent; on wetting again for one month, total expansion 0.13 to 0.17 per cent.

Neat cement bars hardening in air: Contraction after 3 months, 0.14 to 0.28 per cent; after 1 year, 0.18 to 0.34 per cent with a slight increase to 4 years.

Mortar bars of 1:3 mix hardening under water: Expansion 0.01 to 0.05 per cent at 1 year, the more rapid changes occurring during the first few weeks.

Mortar bars of 1:3 mix hardening in air. Contraction 0.06 to 0.09 per cent at one year, most of the contraction being in the first 3 months.

Tests by Raymond E. Davis at the University of California upon concrete bars 3 by 3 by 40-in. of a 1:2:3 mix by volume using three California portland cements indicated rather small variations for the specimens stored continuously in air (0.078 to 0.082 per cent contraction at the age of three months) while the expansions for the specimens stored continuously in water for the same period ranged from 0.007 to 0.011 per cent.

C. A. Wright and H. H. Scofield[50a] of Cornell University made comparative tests of the volume changes occurring in mortars and

concretes made of high alumina cement and of portland cement during the first 24 hours after mixing. The aggregates were a bank sand and screened gravel and the slump of all mixes was 6 to 7 in. The 6 by 6 by 40-in. specimens were cast in smooth watertight metallic molds; the air surrounding the specimen being kept at a temperature of 20° C. and a relative humidity of 60 per cent. Two micrometer microscopes mounted on a brass scale and sighted on brass plugs imbedded in the specimens were used in making the observations. The data from the tests are presented in Table 5.

TABLE 5

EARLY VOLUME CHANGES (WRIGHT AND SCHOFIELD)

Mix	Percentage Length Change During First 24 Hours	
	High Alumina Cement	Portland Cement
1:1 mortar	—0.143	+0.007
1:3 mortar	—0.105	+0.002
1:5 mortar	—0.042	+0.016
1:1½:3	—0.087	—0.006
1:2:4	—0.046	—0.016
1:2½:5	—0.027	+0.008

This table shows that the early volume changes of the alumina cement mixes are considerably greater than for the portland cement mixes.

2. *Proportions of Cement and Aggregate*—The cement ratio has a definite effect upon the volume change; the leaner the mix, the less the change (expansion or contraction) occurring over a period of time. The comparative results as obtained by some early investigators, are presented in Table 6.

F. R. McMillan[27] conducted tests on 4 by 5 by 24-in. bars of limestone concrete of varying proportions. The bars were cured 2 weeks under wet burlap and were then stored in air for 430 days. The net percentage contractions after the first observations at one day follow:

Mix	*Contraction*	*Mix*	*Contraction*
1:1 1/2:3	.066%	1:2 1/2:5	.060%
1:2:4	.064%	1:3:6	.055%

Dr. Morsch[32] has reported tests of the influences of cement ratio and the quality of the sand upon the volume changes in water and

TABLE 6

PERCENTAGE VOLUME CHANGE AS AFFECTED BY CEMENT RATIO (WHITE)

Authority	Neat		1:3 Mortar		Duration of Test	Size of Specimen
	Exp.	Contr.	Exp.	Contr.		
Bauschinger[1]	0.046	0.214	0.009	0.132	16 wks.	4.7-in. cube
Tornei	0.025	0.201	0.027	0.094	12 wks.	4.7-in. cube
Gary[5]	0.026 0.092	0.129	0.018 0.020	0.053	4 wks. 2 yrs.	
Schuman[3]	0.082		0.021		4 wks.	0.77 sq. in.
Schuman[3]	0.150		0.027		5 yrs.	by 3.9 in.
Considere[6]	0.079	0.132	0.028	0.050	9 wks.	
Graf[11 and 36]		0.242		0.111*	2 yrs.	
White[24]	0.111		0.025		3 yrs.	1x1x4-in.
White[24]		0.321		0.072	4 yrs.	1x1x4-in.

*1:2 mortar.

in air. He noted that neat cement and also 1:3 and 1:5 mortar specimens in water swelled continually during the first year, whereas the shrinkage in air reached a maximum at 90 days. The relative expansion at one year in water and the contractions at 90 days in air were:

	Relative Water Expansion	Relative Air Shrinkage
Neat cement.....	180	150
1:5 mortar.....	45	52

Investigations by Goldbeck and Jackson[30] on 8 by 8 by 60-in. bars of mortar and of concrete of various mixes stored in air disclosed no marked difference in volume changes due to the cement ratio, although the lean mixes contracted the least. The rich mortar in water expanded about half again as much as the lean mortar. Their results are presented in Table 7.

In the tests of Matsumoto[33] at the University of Illinois, it was found that a rather direct relationship existed between the cement ratio and the expansion in water as well as the contraction upon oven drying. The specimens were 2 by 3 by 24-in. bars. They were first kept wet for 60 days, then air dried 58 days, and

TABLE 7

EFFECT OF RICHNESS OF MIX (GOLDBECK AND JACKSON)

Mix	Percentage Volume Change		Test Duration Days	Remarks
	Exp. in Water	Contr. in Air		
1:2 mortar	0.025		560	Expansion completed
1:3 mortar	0.016		560	
1:2 mortar		0.11	310	Contr. progressing at rate
1:3 mortar		0.11	310	of .01% in 40 days.
1:2:4 concrete		0.065	800	Results the same for wet
1:3:6 concrete		0.060	800	and dry consistencies.

finally dried in an oven at 200° F. The results are shown in Table 8, the net contractions of the last two columns being based upon the first observation after casting.

TABLE 8

EFFECT OF RICHNESS OF MIX (MATSUMOTO)

Mix	Percentage Expansion Wet—60 Days	Percentage Net Contraction Air—58 Days	Percentage Net Contraction Oven—29 Days
1:1	0.0124	0.046	0.157
1:2	0.0100	0.049	0.124
1:3	0.0101	0.050	0.108
1:1:2	0.0091	0.036	0.081
1:2:4	0.0055	0.033	0.064
1:3:6	0.0019	0.036	0.062

It will be observed that the contractions after 58 days in air do not bear a consistent relationship to the cement ratio, indicating that the results of a short-time test are not always a criterion of values which would ultimately obtain.

The effect of richness of mix is further demonstrated by the results of a comprehensive series of tests by W. K. Hatt[55] at Purdue University which are presented in Table 9. The specimens were 4 by 4 by 48 in. They were cured for 14 days under wet burlap previous to the first drying period. The values for the concrete specimens are averages for three coarse aggregates—gravel, limestone and slag. It is seen that the contractions were greater for the first drying period than for any following period and also that a residual shrinkage after the final soaking is shown for all bars except those of neat cement, even though the weights

are greater than at the beginning. The table also shows that the richer the mix the greater the change in length from one extreme storage condition to the other.

TABLE 9

EFFECT OF RICHNESS OF MIX (HATT)

Mix	Percentage Contraction		Percentage Expansion		Residual Change after 2nd Soaking Period
	1st Drying 182 Days	2nd Drying 255 Days	1st Soaking 130 Days	2nd Soaking 106 Days	
Neat	0.089	0.061	0.092	0.068	+0.010
1:2	0.085	0.050	0.059	0.042	—0.030
1:2:3	0.044	0.026	0.032	0.026	—0.007
1:2½:4	0.040	0.023	0.027	0.022	—0.010
1:2½:5	0.037	0.021	0.024	0.020	—0.014
1:3:6	0.034	0.019	0.021	0.016	—0.012

Tests by Davis and Troxell[56] at the University of California also show greater volume changes for rich mixes than for lean ones. The 3 by 3 by 40-in. bars were of gravel concrete of 4 different gradations and were cured in water for 28 days previous to oven drying. Table 10 shows the data from these tests.

TABLE 10

EFFECT OF RICHNESS OF MIX AND GRADATION OF AGGREGATE (DAVIS)

Concrete	Percentage Contraction, Wet to Oven Dry	
	Aggregate of Low Surface Modulus	Aggregate of High Surface Modulus
1:2	0.117	0.101
1:3	0.105	0.090
1:4½	0.090	0.080
1:6	0.090	0.068

J. C. Pearson[34] of the Bureau of Standards made shrinkage tests of mortars and concretes to determine the effect of richness of mix and also the effect of the absorptive character of base. The initial measurements were made while the specimens were still plastic and observations were continued for 9 weeks. The specimens were 1 by 4 by 24 inches, stored in air. The results are given in Table 11.

3. *Effect of Type and Gradation of Aggregate*—Many investigators have shown the important influence of the type of aggregate upon the volume changes developed in mortars and concretes.

TABLE 11

EFFECT OF RICHNESS OF MIX AND ABSORPTIVENESS OF BASE (PEARSON)

Mix	Percentage Contraction at 64 Days	
	On Non-Absorptive Bases	On Dry or Damp Absorptive Bases
Neat cement	0.217	
1 cement : 1 sand	0.123	
1 cement : 2 sand	0.110	0.067—0.108
1 cement : 3 sand	0.109	0.064—0.067
1 cement : 4 sand	0.094	0.059—0.081
1 cement : 5 sand	0.086	0.060—0.080
1 cement : 6 sand	0.084	0.054—0.063
1 cement : 4 crushed gravel	0.067	0.028
1 cement : 6 crushed gravel	0.050	0.024

In connection with tests to determine the volume changes in neat cements and cement mortars, Schumann[3] observed the expansion and contraction of various building stones when alternately wet and dried. The results are shown in Table 12.

TABLE 12

VOLUME CHANGES IN BUILDING STONES (SCHUMANN)

Kind of Stone	Percentage Change in Two Weeks	
	In Water	In Air
Sandstone—red, fine grained	+0.006	—0.018
red, coarse	+0.016	—0.023
green, fine grained	+0.050	—0.050
red, fine grained	+0.050	—0.050
red, very fine grained	+0.206	—0.178
Limestone—white	+0.004	—0.008
Lothringer Lias	+0.007	—0.008
Challe	+0.011	—0.009
dense, containing clay	+0.026	—0.026
Granite	+0.006	—0.015
Basalt —a	+0.026	—0.027
b	+0.041	—0.050
c	+0.048	—0.057
d	+0.023	—0.057

Matsumoto[33] subjected 3 by 4 by 24-in. bars of sandstone and limestone to moisture expansion tests to determine the volume changes in the stone acting alone. The bars were first oven dried 6 days and then soaked 13 days. The sandstone absorbed 5.75 per cent water and expanded 0.080 per cent, or somewhat more than that of ordinary concrete. Of this amount, 0.070 per cent

expansion occurred the first day. Limestone treated in a similar way expanded only 0.001 per cent but it absorbed 6 per cent of water.

Cloyd M. Chapman[44] made several tests on 2 by 2 by 24-in. bars to determine the effect of type of fine aggregate upon the contraction of 1:2 mortars and 1:2:4 concretes. The fine aggregates were all of the same gradation and the coarse aggregate for all the concrete specimens was a ¾-in. gravel. The contractions after 7 days in water and 1 year in air are shown in Table 13.

TABLE 13

EFFECT OF TYPE OF FINE AGGREGATE UPON AIR SHRINKAGE (CHAPMAN)

Fine Aggregate (Screenings)	Percentage Contraction	
	1:2 Mortar	1:2:4 Concrete
Conglomerate	0.21	0.075
Trap Rock	0.21	0.093
Slag	0.14	0.068
Granite	0.13	0.068
Limestone	0.13	0.062

In further tests Chapman[44] determined the effect of gradation of the conglomerate upon the contraction of a 1:2 mortar by screening out all particles under the 20 mesh. The mortar with fine particles removed contracted only 0.18 per cent as compared with 0.21 per cent for the normal aggregate.

Tests by F. R. McMillan[27] on 4 by 5 by 24-in. bars of 1:2:4 concrete show similar variations in the volume changes resulting from the use of different aggregates. The contractions at 420 days for sandstone, trap rock and granite aggregates were respectively 0.066, 0.063 and 0.035 per cent. No data are presented to show the effect of gradation of the aggregates. Observations were begun at the age of two days, the specimens having previously been covered with wet burlap.

The effect of various coarse aggregates upon the volume changes of 4 by 7 by 48-in. beams was determined by Hatt and Mills.[55] The same type of sand was used as the fine aggregate for all beams. The average changes for 4 mixes ranging from 1:2:4 to 1:3:6 are presented in Table 14. The specimens were cured under wet burlap for 17 days previous to the first air-drying

period, after which they alternated between air-dry storage and immersion in water for periods shown in the table.

TABLE 14

EFFECT OF TYPE OF COARSE AGGREGATE UPON VOLUME CHANGES (HATT AND MILLS)

Coarse Aggregate		Percentage Contraction		Percentage Expansion	
Type	Fineness Modulus	1st Drying Period—182 Days	2nd Drying Period—255 Days	1st Soaking Period—130 Days	2nd Soaking Period—106 Days
Gravel	6.68	0.046	0.025	0.027	0.023
Slag	7.72	0.038	0.021	0.028	0.021
Limestone	7.88	0.033	0.021	0.023	0.020

These data agree with those of Tables 12 and 13 in showing a smaller volume change for limestone concrete than for any of the others, although other tests by Hatt and Mills show that the contraction of a soft, porous limestone concrete may exceed that for a gravel concrete. It is significant that in Table 14 a greater length change is recorded for the first drying period than is found in any of the periods following.

Raymond E. Davis, in tests at the University of California, determined the effect of the type of aggregate upon the volume changes of 3 by 3 by 40-in. specimens of concrete made of a 1:2:3 mix by volume and using a water-cement ratio of 0.90. For all of the specimens except those of gravel, the fine aggregate was obtained by crushing some of the coarse aggregate to the proper size. The gradation of the mixed aggregate was the same for all materials, its fineness modulus being 5.58. The results of the tests follow in Table 15.

TABLE 15

EFFECT OF TYPE OF AGGREGATE (DAVIS)

Aggregate	Percentage Change in Three Months	
	Contraction in Air	Expansion in Water
Gravel	0.079	0.0074
Sandstone	0.075	0.0055
Limestone	0.039	0.0050
Granite	0.037	0.0131
Quartz	0.036	0.0094

These results again demonstrate the small contraction of granite and limestone concretes in comparison with similar gravel and sandstone concretes. The results also show that the magnitude of expansion of concrete in water is no indication of what the shrinkage will be when the concrete is exposed to dry air, the granite and quartz concretes expanding the most in water yet contracting the least in air.

The fact that mortar mixes of a given cement ratio are subject to greater volume change than concrete mixes of the same cement ratio (see Tables 8 and 11) has given rise to the general impression that the changes in length of mortar and concrete specimens vary directly with the fineness of the aggregate. This appears to be further substantiated by the tests of Chapman, described previously, in which the removal from the conglomerate screenings of the material passing the 20-mesh sieve, reduced the contraction in air from 0.21 to 0.18 per cent.

Certain tests by Hatt[55] were conducted to determine the effect of the gradation and character of the coarse aggregate upon the volume changes of 4 by 7 by 28-in. specimens of a 1:2:3 mix stored 14 days under wet burlap followed by 399 days in air and 185 days in water. The same kind of sand (a well-graded, washed aggregate having a fineness modulus of 3.50) was used for all specimens, but the coarse aggregate for one series of bars was a soft limestone, while gravel was used for a second series. Three groups of specimens were made from different gradations of these coarse aggregates. The results presented in Table 16 showing the effect of these three gradations of coarse aggregates are the averages for the two types of aggregates used. It will be noted that all changes are greatest for the mix of finest gradation and that the permanent shrinkage after re-immersion is also greatest for that mix. It would have been of special interest to have determined the effect of variation in gradation of the fine aggregate also.

In order to determine the effect of the finer particles of a granite aggregate upon shrinkage and expansion, Davis and Troxell[56] conducted two series of tests upon 3 by 3 by 40-in. specimens for which the aggregate was composed entirely of granite under ¾-in. in size. For both series the water-cement ratio was 1.0. For the first series the cement ratio was kept con-

TABLE 16
EFFECT OF GRADATION OF COARSE AGGREGATE (HATT)

Large Aggregate	Fineness Modulus	Length Change in Per Cent			
		14 Days Wet Burlap	399 Days in Air	185 Days in Water	Final Permanent Shrinkage
Coarse....	7.15	+0.0040	—0.055	+0.038	—0.012
Medium...	6.70	+0.0048	—0.065	+0.040	—0.020
Fine.....	6.25	+0.0048	—0.067	+0.041	—0.022

stant, the mix being 1:4 and the slump varying from 4 to 6 in. depending upon the percentage of fine material. In the second series, the slump was kept constant at 3 in., the mix varying from 1:4.5 to 1:5.2 depending upon the per cent of fine material. There were three groups within each series, the percentage of the fine aggregate passing the 100-mesh sieve for the three groups being 0.0, 6 and 25 per cent. For all mixes, the volumetric ratio of fine to coarse aggregate was 2 to 3. After 4 days curing under wet sacks, all specimens were stored in air for 2 weeks, oven-dried, immersed in water for 1 year, stored in air for 5 months, oven-dried a second time and then stored in air for 9 months. These tests disclosed that the percentage of fines had no marked influence either upon the magnitude of the shrinkage in the oven-dry state or upon the magnitude of the expansion after the prolonged period of immersion.

4. *Effect of Admixtures of Lime, Clay, Etc.*—The influence of lime admixtures upon the expansion of 4 by 4 by 24-in. bars of 1:5 (by weight) bank-run gravel concrete and of 1:5 bank-sand mortar caused by immersion for 24 hours subsequent to about 7 weeks in air was the subject of a report by Scofield and Stinchfield.[29] For each series, the amount of lime was varied from 0 to 15 per cent of the cement by weight. The expansions observed in the concrete series for 0.0, 7.5 and 15 per cent of lime were 0.017, 0.018 and 0.020 per cent respectively. The bars of the mortar series containing 0.0, 7.5 and 15 per cent of lime expanded 0.019, 0.022 and 0.022 per cent respectively. These results indicate that the lime admixture had no appreciable effect upon the resulting expansions. The kind of lime is not reported.

Davis and Troxell[56] also investigated the volume changes of cement-lime mortar bars 3 by 3 by 40-in. in length. For all specimens the basic mix was one volume of portland cement to 3 volumes of a fine river sand. To the basic mix were added various percentages of high calcium hydrated lime, so that for the 5 groups of specimens, the lime content varied from zero on the one hand up to a volume equal to that of the cement on the other. The consistency of all the mortars was such as to produce a slump of about 5-in. At the end of 28 months storage in dry air, the total shrinkage of the various groups varied from 0.15 to 0.16 per cent. There was no evidence that the addition of the lime to the basic cement-mortar mix appreciably altered the shrinkage, though the contraction was least for the plain cement mortar.

Some bars of each of the 5 groups of the preceding series were immersed in water after 7 months air storage. Within 20 hours after immersion the bars had recovered about one-half the total shrinkage that had developed during the preceding 7 months in air. After all changes had ceased, there was still a net shrinkage of from 0.04 to 0.06 per cent or about one-third that which took place in air prior to immersion. The expansion following immersion was greatest for the mortar with largest lime content and, in general, least for the mortar without lime; but the difference was not large and there is lack of consistency in this respect when all groups are considered.

Table 17 presents the results of a second series of tests by Davis and Troxell. The 1:3 and 1:5 mortars proportioned by volume included various admixtures but were all of the same consistency as measured by a flow table. The percentage by weight of hydrated high magnesium lime and of clay shown in Table 17 replaced a corresponding weight of cement—that is, they were not additional to the regular 1:3 or 1:5 mix. One group of specimens for each mix was cast in absorbent wooden molds which were removed after 48 hours, a second group was cast in absorbent burned clay molds which were likewise removed after 48 hours, while a third group was cast in absorbent burned clay molds which were permanently left on. All specimens were continuously stored in dry air for 375 days. A study of the tabulated data for the specimens with molds removed shows that the replacement of 10 per cent of the cement by hydrated high mag

nesian lime reduces the contraction somewhat; whereas the replacement by clay increases the contraction as does also the use of a so-called plastic, waterproof cement. The effect of the absorption of the mold is to reduce the shrinkage materially.

TABLE 17

EFFECT OF ADMIXTURES AND SURROUNDING MEDIUM (DAVIS AND TROXELL)

Mortar	Percentage Contraction, 375 Days in Air		
	Non-Absorbent Wood Molds Removed	Absorbent Burned Clay Molds Removed	Absorbent Burned Clay Molds Left On
Plain 1:3 mix	0.119	0.102	0.072
10 per cent clay in 1:5 mix	0.092	0.109	0.087
10 per cent hydrated lime in 1:3 mix	0.106	0.094	0.080
10 per cent clay in 1:3 mix	0.133	0.122	0.095
5 per cent clay in 1:3 mix	0.135	0.114	0.085
Plastic cement 1:3 mix	0.153	0.134	0.096

Tests by White[14] and by Davis and Troxell[56] indicate that the use of integral waterproofing compounds (at least those investigated) does not aid in reducing the volume changes resulting from alternate wetting and drying of cement mortars.

5. *Effect of Consistency of Mix*—Davis and Troxell[56] report two series of tests in progress—one series for mortars and one for concretes—to determine the effect of quantity of mixing water upon volume changes. The observations began at the age of about 2 days. The test results presented in Tables 18 and 19 show that for air storage the wet, rich mixes contract more than the dry ones, but the effect upon the lean mixes is much less pronounced. For the specimens in water storage, the dry mixes in general expand the most, although the lean mixes show but little variation for the various consistencies.

The tests by Goldbeck and Jackson[30] to ascertain the effect of consistency of the mix upon the contraction in air showed practically no difference due to this factor. Their tests were made on very wet and very dry mixes of 1:2:4 and 1:3:6 concrete at ages up to 800 days.

McMillan[27] obtained different results for air storage. Specimens 4 by 5 by 24 inches in length were made of 1:2:4 concrete of four consistencies, the water content ranging from 7.75 to 10.0 per

TABLE 18

EFFECT OF WATER RATIO ON CONCRETES (DAVIS AND TROXELL)

Concrete Mix	Water Cement Ratio	Percentage Change in 3 Months	
		Expansion in Water	Contraction in Air
1:4	0.98	0.0184	0.068
	1.07	0.0204	0.072
	1.16	0.0165	0.080
1:6	0.98	0.019	0.064
	1.07	0.019	0.064
	1.16	0.018	0.069

TABLE 19

EFFECT OF WATER RATIO ON MORTARS (DAVIS AND TROXELL)

Mortar Mix	Slump Inches	Percentage Change in 3 Months	
		Expansion in Water	Contraction in Air
1:2	0	0.036	0.105
	2	0.024	0.110
	4	0.023	0.138
1:6	0	0.014	0.078
	2½	0.016	0.085
	9	0.013	0.082

cent. All specimens were cured two weeks under wet burlap and were then stored in dry air. At 430 days, the shrinkage varied directly with the amount of water used in the mix and ranged from 0.055 to 0.063 per cent.

6. *Effect of Moisture Conditions*—Most of the early investigations of volume changes of cement were made as a part of the soundness or constancy of volume test. They were generally made upon neat cement or a rich mortar and were usually immersed in water, oftentimes at a high temperature. The tests were later extended to cover air storage and include concretes as well as mortars. Storage conditions for volume change tests may include immersion in hot or cold water, a covering of wet burlap or damp sand, dry or moist air, all at various temperatures. However, in recent carefully conducted tests, the temperature and moisture conditions have been accurately controlled in order to eliminate any uncertainties of test conditions.

Many of the tests discussed earlier in this review (see Tables 2, 6, 7, 8, 9, 12, 14, 15, 16, 18 and 19) show conclusively the effect of conditions of storage upon the resulting length changes, contraction occurring in dry air and expansion in water. The action of concrete in damp air depends upon the dryness of the specimen at the time of the test; if dry it will expand and if wet it will contract. The results of various measurements, therefore, are indicative of ranges for a particular condition and so cannot be applied directly to any other specified case.

Tables 9 and 14 summarizing tests by Hatt[55] show that the previous exposure and the resulting condition of the specimen influence the later changes. The first change of mortars or concrete from wet to dry was always found to be greater than the subsequent changes. This same effect was observed by Davis and Troxell[56] in connection with tests of gravel and granite concretes, and also by Goldbeck and Jackson.[30]

However, White[14] found that for neat cement and rich mortar bars, there appeared to be a progressive growth of the specimen due to alternating the wet and dry storage conditions. He explains this by stating that thorough drying of the specimen opens up certain passages through the mass due to shrinkage and that the water may penetrate more deeply into the cement grains upon re-immersion and that therefore a greater length than the original immersed length obtains. It is quite probable, however, that very fine cracks develop in the bar due to the excessive drying and that particles of dust which may enter the cracks may cause a wedging action upon soaking and hence increase the length of the specimen. White[21] states that the length of a bar which has passed through a cycle of drying and wetting is not usually the same as it was before the cycle, but it may be longer or shorter depending upon the actual treatment received.

Tests have been made by some investigators to determine the effect of the initial curing upon the later volume changes which occur while the concrete dries out. Goldbeck and Jackson cast specimens of 1:2:4 and 1:3:6 mixes and of both wet and dry consistencies. Some of these specimens were placed in dry air immediately whereas others were first carefully cured in water for 14 days previous to exposure to the dry air. These tests disclosed but slight benefit resulting from the water-curing period, as the

rich and lean mixes so treated contracted 0.063 and 0.055 per cent respectively whereas the corresponding specimens dried immediately upon casting contracted 0.065 and 0.060 per cent. Immersion for a period of six months also failed to reduce the later contraction resulting from air drying.

McMillan[27] also concluded that damp curing of the concrete has no effect upon the contraction resulting from long storage in dry air. He prepared 4 by 5 by 48-in. specimens of 1:2:4 concrete containing 0.3 per cent of reinforcement. All specimens were cured under wet burlap for three days after which one was left uncovered, a second was alternately wet and covered but was finally left uncovered after 180 days, while a third was kept wet for 12 weeks and was then uncovered. The first shrank the most rapidly, the second contracted less rapidly and the third shrank the least rapidly. However, after one year all of the specimens had contracted the same amount (0.08 per cent), thus showing that the ultimate total shrinkage resulting from air drying is independent of the method of curing.

Mörsch[32] found for 1:3 mortar bars that the length of the initial period of water curing affected the final net contraction resulting from exposure to dry air based upon an initial observation of length at the time the specimen was cast. The specimens were immersed in water for 0, 7, 28, 90 and 196 days and were then allowed to dry in the air until they were 4 years old. The results are shown in Table 20.

TABLE 20

EFFECT OF MOIST CURING ON AIR SHRINKAGE (MORSCH)

Water Curing, in Days	Percentage Volume Change		
	Expansion in Water	Final Contraction in Air	Actual Contraction, Wet Cured Length to Dry
0		—0.032	—0.032
7	+0.010	—0.028	—0.038
28	+0.017	—0.022	—0.039
90	+0.027	—0.004	—0.031
196	+0.032	+0.004	—0.028

The actual contraction shown in the last column is that which occurred between the time of reaching the maximum expansion

shown in column two and the final observation at the age of 4 years.

The above data seem to show that the actual contraction of mortar after the period of curing is more or less independent of the magnitude of the previous expansion resulting from curing in water and indicate that the final contraction (based upon the original length) is inversely related to the expansion developed by the water curing; that is, those which expand the most during curing, develop the lesser net contraction in air.

Hatt[47] states that the careful curing of concrete beginning at an early age will postpone the period of surface shrinkage until the concrete has attained sufficient strength to take up this shrinkage without experiencing cracks. Evidence also indicates that the presence of small amounts of reinforcement near the surface will preserve the integrity of the surface of concrete under favorable curing conditions.

For thoroughly dried concrete bars, Davis and Troxell[56] found that air of high relative humidity was nearly as effective in producing expansion as was immersion in water.

Goldbeck and Jackson[30] found that the contraction of concrete in dry air could be effectively reduced by coating the concrete with tar so that the contained moisture might not escape. Blocks of 1:2:4 concrete 8 by 8 by 60 in. in length were water-cured for 13 days at which time their expansion amounted to 0.004 per cent. The specimens were then air dried for one day, after which they were coated with tar. The bars continued to increase in length reaching an expansion of 0.006 per cent at 30 days. Contraction then began and amounted to 0.003 per cent at 210 days whereas for a specimen without the tar coating the normal contraction would have amounted to 15 or 20 times as much.

7. *Effect of Age at Time of First Measurement*—The first measurement to determine the volume changes occurring in cement mortars and concretes has ordinarily been made after the specimen has acquired sufficient strength and rigidity so that it may be safely handled, usually at the age of one or two days. Since mortars and concretes expand when stored in water and contract when stored in air it appears possible that if the proper amount of moisture is supplied, no change in length will occur. Curing under damp burlap is an attempt to furnish just enough

moisture so that this condition will be realized—how closely is problematical. However, the friction of the forms of a small test specimen offer considerable resistance to any change of volume so that it is questionable if in the usual laboratory test any appreciable volume change occurs prior to the removal of the molds at the age of one or two days, provided that no considerable moisture is added or taken away from the mass.

Tests made by J. C. Pearson[34] at the Bureau of Standards in 1921 were designed to determine the volume changes in mortar mixes from the time that the 1 by 4 by 24-in. specimens were molded. Although an attempt was made to eliminate the resistance of the forms to volume changes in the mortars, it is questionable whether the specimens were perfectly free to move. The experiments indicated either moderate expansion or contraction for the first 4 hours while the mortars were still plastic. During the initial set, a pronounced contraction usually resulted but the rate of shrinkage was reduced at 8 to 10 hours during the time of final set. Very little change occurred from 12 to 24 hours but thereafter a long continued shrinkage began, the various cements showing about the same rate of change after 24 hours, although the difference up to that time was marked. Table 11 shows the total contractions at the age of 64 days for several mixes cast on non-absorbent bases. Undoubtedly a large part of the 24-hour contractions observed by Pearson were caused by loss of moisture due to evaporation. Pearson concluded that the initial volume changes could be reduced and controlled within fairly close limits by taking the necessary precautions.

Graf[36] also investigated the magnitude of the early volume changes in cement mortars stored in air. The material was cast in the form of discs 15 mm. thick and 150 mm. in diameter placed on oiled paper. Movements were observed with a micrometer microscope. He observed rather large contractions during the first 9 hours (particularly for the rich mixes) in comparison with the later changes, but the thinness of his specimens must be considered in the interpretation of his test data. Some of Graf's results are shown in Table 21. The last column gives the shrinkage at 28 days, based upon an initial observation at 24 hours.

TABLE 21

SHRINKAGE OF THIN DISCS (GRAF)

Mix	Percentage Contraction in Air		
	0 to 9 Hours	0 to 7 Days	1 to 28 Days
Neat cement	0.42	0.52	0.13
1:1 mortar	0.32	0.38	0.08
1:4 mortar	0.05	0.13	0.07

8. *Effect of Duration of Test*—Expansion or contraction continues for an extended period although for constant conditions of storage the change in length after a year or two is small in comparison with that developed previously. The variation in reported results is partially due to many tests not being continued long enough for volumetric equilibrium to be reached. The effect of the duration of the test upon volume changes of 1 by 1 by 4-in. neat cement bars has been investigated by White,[14] with the results shown in Table 22.

TABLE 22

EFFECT OF THE DURATION OF TEST ON VOLUME CHANGES (WHITE)

Duration of Test	Percentage Change in Length	
	Under Water	In Air
7 days	+0.030	—0.085
2 months	+0.080	—0.225
6 months	+0.090	—0.285
1 year	+0.100	—0.303
5 years	+0.144	—0.381
10 years	+0.162	—0.410
20 years	+0.161	—0.427*

*Air dried by Calcium chloride.

Tests conducted by White [14] and others show that up to ages of at least 20 years mortars and concretes still respond quickly to a variation in moisture conditions, although the rate of change in length may not be quite as great as for a freshly cast specimen.

9. *Effect of Size and Shape of Specimen*—As the size and shape of the specimen have a marked effect upon the rate of loss of moisture upon exposure to dry air or upon the gain of moisture upon immersion, it is evident that these factors will exert their influence upon the rate of volume change as well as upon the total expansion or contraction. There is also greater likelihood of un-

equal distribution of moisture in large, thick specimens than in thin ones. The frictional restraint of the forms, the bending of the specimen on its supports during observation and the relative precision of the measurement all depend upon the type of specimen. Moreover, the size of the specimen will affect the formation of surface cracks which will have considerable influence upon subsequent volume changes. The specimens in the investigations here considered range in size from 5-in. cubes to prisms varying from 1 by 1 by 4 in. to 8 by 8 by 108 in.; consequently the results are not directly comparable.

10. *Effect of Method of Mixing and Placing and Absorptiveness of Forms*—It is conceivable that any factor affecting the porosity of the mass might exert some influence upon the absorption or loss of moisture taking place with time and thus modify the resulting volume changes.

McMillan[27] observed that small variations in density of the mix made no appreciable difference in the volume changes.

Pearson[34] has determined that absorptive forms will greatly reduce the shrinkage of mortar specimens. As shown in Table 11, the total normal shrinkage (including the plastic shrinkage) for the specimens cast on non-absorptive bases may be reduced on the average from 25 to 50 per cent by the use of a dry or damp absorptive base.

Tests by Davis and Troxell[56] (see Table 17) likewise show a contraction about 15 per cent less for 1:3 mortars cast in absorbent molds than for similar mortars cast in non-absorbent molds. The first observations for these tests were made at the age of 2 days, all bars being covered by damp sacks up to that time.

Graf[36] reports that cast iron ends in the molds decreased the normal contraction of the mortar.

11. *Effect of Amount and Distribution of Reinforcement.*—Steel reinforcement decreases the expansion and contraction of mortar and concretes. When shrinkage occurs in an unrestrained reinforced structure, compressive stresses are developed in the steel and tensile stresses are set up in the concrete. Upon expansion of the mass, tension exists in the steel and compression in the concrete.

Considére,[6] in 1899, reported the behavior of small bars of neat cement and of 1:2.5 mortar. The prisms were 1 by 2.4 by 24 in. Half of them were reinforced with a round steel bar 0.4 in. in diameter. The measurements extended over a period of 63 days and at the close of the period showed the following changes:

TABLE 23
EFFECT OF REINFORCEMENT (CONSIDERE)

Type of Specimen	Per Cent Change in Length	
	In Water	In Air
Neat cement, not reinforced	+0.079	—0.132
Neat cement, reinforced	+0.022	—0.025
Mortar, not reinforced	+0.028	—0.050
Mortar, reinforced	+0.006	—0.010

Matsumoto[33] conducted 3 sets of tests upon 1:2:4 reinforced concrete to determine the effect of steel reinforcement upon the contraction and to determine the shrinkage stresses developed in the steel and concrete. For Series 1, the specimens were 2 by 3 by 24 in., the reinforced specimens having 2 longitudinal steel bars symmetrically placed. They were stored 22 days in air followed by 13 days in an oven at 150° F. For Series 2, the specimens were 6 by 6 by 24 in., the reinforced specimens having 4 steel bars, one near each corner. They were stored 99 days in air. The specimens of Series 3 were similar to those of Series 2 except that slippage of the bars was prevented by lugs welded to the reinforcement. The data presented in Table 24 show that the contraction is materially reduced by the introduction of steel bars into the mass, but high tensile stresses are developed in the concrete, the more the reinforcement, the greater the stress in the concrete. In fact, further drying of the bars in an oven at 200° F. caused such high tensile stresses in the concrete that cracks re-resulted.

It is shown by Matsumoto's tests that, in reinforced concrete, the shrinkage stress in the steel may reach the safe working stress if there is less than 1.5 per cent of steel and that the shrinkage stress in 1:2:4 concrete may exceed the ultimate tensile strength of the concrete if the reinforcement exceeds 1.5 per cent.

Experimental studies were conducted by Bach and Graf[11] on bars 62 sq. in. in section and 39 in. long made of a 1:3:2 mix, 4

TABLE 24
EFFECT OF REINFORCEMENT (MATSUMOTO)

Series	Percentage Reinforcement	Percentage Contraction	Shrinkage Stress, lb./sq. in.	
			Steel, Comp.	Concrete, Tension
1	0.00	0.061	0	0
	3.68	0.017	5,400	190
	6.54	0.012	3,500	240
2	0.00	0.064	0	0
	0.55	0.054	16,400	90
	1.23	0.042	12,500	150
	2.18	0.029	8,500	190
3	0.00	0.064	0	0
	0.55	0.054	19,000	100
	1.23	0.042	15,500	200
	2.18	0.029	12,000	250

bars being of plain concrete while 4 others were each reinforced with one steel bar having a cross-sectional area of 0.49 sq. in. All bars were cured several days under wet cloths. Two bars of each group were then stored continuously in air, and the remaining bars were stored in water. The changes after six years were very slight but the observations were continued for 12½ years at the end of which time the length changes were as follows:

TABLE 24A
EFFECT OF REINFOPCEMENT (BACH AND GRAF)

Type of Specimen	Percentage Change	
	Expansion in Water	Contraction in Air
Plain concrete	0.020	0.052
Reinforced concrete	0.009	0.022

In connection with studies of concrete experimental highways, J. T. Pauls[43] observed that "with a high percentage of continuous steel, relatively fine, closely spaced cracks may be looked for; with a low percentage, breaks in the steel may be expected to permit wider cracks to form at considerable intervals. There is possible danger in the use of too high a percentage of longitudinal steel, for under such conditions, numerous fine traverse cracks will develop and there is the possibility that the narrow transverse beams thus formed will crack under traffic."

Specimens of 1:2:4 gravel concrete, prepared by Goldbeck and Jackson[30], were reinforced with varying amounts of steel. The specimens were 8 by 8 by 60 in. After 6 months storage in dry air, bars of plain concrete had contracted 0.035 per cent; bars containing 0.6 and 1.2 per cent of reinforcement steel had contracted 0.02 per cent; whereas bars containing 1.8 per cent of reinforcement contracted only 0.01 per cent. The theoretical tensile stresses in the concrete ran fairly high (750 pounds per square inch) but no cracks were visible in the concrete. It is probable that slow flow within the concrete took place under the extremely gradually applied load, making the induced tensile stresses very much smaller than the calculated amount.

A series of 1:2 mortar beams 4 by 4 in. in section, some plain and some reinforced, with 0.33 per cent of mesh reinforcing, were observed for comparative shrinkage and the results are reported by Hatt.[47] One group of the specimens was stored under wet burlap while a second group was kept in a current of warm, dry air. The length changes at the age of 28 days were as follows:

TABLE 24B

EFFECT OF REINFORCEMENT (HATT)

Storage	Per Cent Change in Length	
	Plain	Reinforced
Wet burlap	+0.011	+0.010
Dry air	—0.045	—0.040

VOLUME CHANGES DUE TO TEMPERATURE VARIATIONS

It is frequently considered that the thermal coefficient of expansion for concrete is not greatly different from that of carbon steel (0.0000065 per 1° F.), 0.0000055 per 1° F. being a commonly accepted value which is regarded as constant for all concretes within the ordinary ranges of temperature. Tests indicate, however, that there is a wide range in the value of the coefficient. Table 25 includes the data of Keller[4a] as well as the results of a review by Shitkewitsch[19] regarding the coefficient of expansion of various cement mortars, concretes and other building stones.

Rudeloff and Sieglerschmidt[18] state that the coefficient of expansion for concrete increases with its age for the first several

TABLE 25

THERMAL COEFFICIENT OF EXPANSION

Material	Coefficient of Expansion for 1° F.	Authority
Neat cement	0.0000068	Meier
Neat cement	0.0000081	Meier
Neat cement	0.0000079	Adie
Neat cement	0.0000059	Bouniceau
Neat cement	0.0000076	Hyatt
Neat cement	0.0000078	Hyatt
Neat cement	0.0000070	Keller
1:3 Mortar	0.0000066	Bouniceau
1:1 Mortar	0.0000061	Keller
1:2 Mortar	0.0000056	Keller
1:4 Mortar	0.0000058	Keller
1:6 Mortar	0.0000051	Keller
1:8 Mortar	0.0000053	Keller
1:2:4 Bedford Limestone concrete	0.0000055	Pence
1:2:4 Kankakee limestone concrete	0.0000056	Pence
Granite (average)	0.0000045	Bouniceau, Adie, Hurst, Dana
Limestone (average)	0.0000051	Bouniceau, Dana, Pence
Marble (average)	0.0000042	Bouniceau, Adie, Ganot
Sandstone (average)	0.0000069	Haswell, Ganot, Dana, Adie, Thurston

months of its life but that for practical purposes an average value of 0.0000055 for 1° F. is sufficiently accurate. They report that concretes which had been very recently made offered some exceptions to the above figure, samples of ages less than 9 days hardened under water showing contraction instead of expansion when heated to 50° C. All samples which had been in air expanded when heated, as did also those kept more than 9 days in water.

Tests by Rae and Dougherty[9] at Columbia University on specimens 4 by 4 by 36 in. made of a 1:2 mortar mix and of a 1:3:5 gravel concrete mix at the age of 5½ years gave a value of 0.0000056 per 1° F. for the mortar specimens and 0.0000065 per 1° F. for the concrete bars.

Pence[8] determined the coefficients for 1:2:4 mixes of broken limestone and gravel concrete, the specimens being 4 in. in diameter and 3 ft. long. A temperature range of 65° F. was produced by placing the specimens in a steel cylinder and then introducing steam so that it is possible that some expansion was

produced by moisture of the steam. The coefficient per degree Fahrenheit for the limestone concrete ranged from 0.0000052 to 0.0000057 while the gravel concrete averaged 0.0000054. A specimen of Kankakee limestone gave a value of 0.0000056.

Mills,[31] in 1916, made tests to determine the thermal expansion of 2 by 4 by 8-in. specimens. The age of the test pieces ranged from 1 to 8 months and the temperature within the electric oven in which the specimens were heated ranged from about 70° F. to 212° F. The coefficient ranged from 0.0000078 for neat cement to 0.0000058 for 1:5 mortar. For concrete it ranged from 0.0000068 for a 1:1.5:3 mix to 0.0000054 for a 1:3:6 mix. These tests indicate that the thermal coefficient of expansion of mortars and concretes increases with the richness of the mix but that the range of values between a very lean concrete and neat cement is comparatively small.

The variation in the coefficient for various neat cements is shown in Table 26 which summarizes the results of tests by Hatt.[50] It will be noted that the coefficients for the wet specimens tested at 2 months are in general lower than for the dry specimens tested at 4 months, but the difference is small.

TABLE 26
THERMAL COEFFICIENT OF EXPANSION PER 1°F. FOR VARIOUS NEAT CEMENTS (HATT)

Brand of Cement	Age, 4 Months in Air Range, 50° F.-73° F.	Age, 2 Months in Water Range 52° F.-90° F.
1	0.0000096	0.0000094
2	0.0000094	
3	0.0000092	0.0000087
4	0.0000090	
5	0.0000089	
6	0.0000083	
7	0.0000085	0.0000085

Other results obtained by Hatt[50] for concretes of various mixes, ages and storage conditions are presented in Table 27. For all groups the coefficient increased steadily with an increase in temperature.

Tests by W. M. Wilson[52] of the effect of a temperature range from 0° to 85°F. upon a reinforced concrete arch bridge at the age of 3 years gave values from 0.0000046 to 0.0000052 per 1° F. for the ribs, and a value of 0.0000056 for the concrete deck. The type of aggregate and the cement ratio were not stated.

TABLE 27

THERMAL COEFFICIENT OF EXPANSION OF CONCRETE (HATT)

Group	Beams Ins.	Mix	Age in Months	Condition	Coefficient Range	Temperature Range, Deg. F.
1	6x8x48	1:1½:3	2	Oven dry	0.0000040 to 0.0000062	65° to 150°
2	4x7x48	1:2:3	2	Wet immersed	0.0000040 to 0.0000056	45 to 95
3	4x7x48	1:2:3½	9	Air dry	0.0000045 to 0.0000057	25 to 75
4	4x7x48	1:2:3	2	Air dry	0.0000049 to 0.0000070	20 to 95

Comprehensive thermal expansion tests of concrete were conducted by Davis and Troxell at the University of California during 1927 and 1928. The first series of these tests was on concrete of a 1:4 mix, the aggregate consisting entirely of granite. Half of the 3 by 3 by 4-in. specimens were stored in water continuously, and the remainder were cured for 28 days in damp sand and then stored in dry air thereafter. The temperature range for the tests was from 39° to 125° F. The data of Table 28 show that there is no marked variation in the coefficient with age or moisture condition. The variation with temperature was also insignificant. It is worthy of note that the observed coefficients are only about two-thirds of the value commonly assumed, although these low values seem logical in view of the low coefficient of thermal expansion of granite itself.

TABLE 28

THERMAL EXPANSION OF 1:4 GRANITE CONCRETE (DAVIS AND TROXELL)

Condition	Coefficient of Thermal Expansion per 1° F.	
	At Age of 7 Weeks	At Age of 4 Months
Immersed in water	0.0000045	0.0000041
In dry air	0.0000038	0.0000039

A second series of tests by Davis and Troxell included both mortars and concretes in both 1:3 and 1:8 mixes (by weight). The aggregate was a graded sand for the mortar tests and a graded gravel for the concrete. Two consistencies as represented by volumetric water-cement ratios of 1.31 and 1.89 were used. Half of the specimens in each group were kept wet continuously, and the remaining specimens were moist-cured for 10 days and

thereafter stored in dry air. Tests were run at the ages of about six weeks and four months, the temperature ranging from 40° to 130° F. Observations were made for both increasing and decreasing temperatures so as to eliminate errors due to possible moisture changes of the dry specimens. The results of these tests are shown in Table 29. From a study of this tabulation it may be observed that the coefficient for wet storage is slightly greater than that for dry storage but there is practically no variation between the different consistencies, ages or mixes. The coefficients were quite constant throughout the entire temperature range.

TABLE 29

THERMAL EXPANSION OF MORTARS AND CONCRETES (DAVIS AND TROXELL)

	Mortar Mixes	Concrete Mixes	Average for Mortar and Concrete Mixes
Wet storage	0.00000476	0.00000495	0.00000486
Dry storage	0.00000468	0.00000443	0.00000455
1.31 Water-cement ratio	0.00000472	0.00000475	0.00000474
1.89 Water-cement ratio	0.00000473	0.00000466	0.00000470
Age 6 weeks	0.00000479	0.00000473	0.00000476
Age 4 months	0.00000470	0.00000469	0.00000470
1:3 Mix	0.00000478	0.00000462	0.00000470
1:8 Mix	0.00000459	0.00000481	0.00000470

For a third series of tests by Davis and Troxell, six groups of specimens were made. The specimens of the first group were of gravel concrete, using three cement ratios (1:3, 1:4½ and 1:6 by weight) and both a 2-in. and 6-in. slump for each richness of mix. Each of the 5 remaining groups of specimens in this series was made of a different aggregate, each aggregate having the same gradation as the gravel of the first group. The aggregates used were quartz sandstone, granite, basalt and limestone, each of these being used in a 1:4½ mix with a 2-in. slump. All of the bars were damp-cured for 16 days after which they were stored in air. After observing their coefficient of thermal expansion between 50° and 130° F., at the age of 9 weeks, half of the specimens of each group were stored in dry air at 70° F., while the remaining bars were subjected to 100 cycles of temperature ranging from 15° to 130° F., after which all of the specimens were again observed for their coefficient of thermal expansion to determine how the

alternating temperatures had affected the values in comparison with the coefficients for the specimens stored at constant temperature. The results of this series may be summarized as follows:

The coefficient increases with the richness of the mix and is slightly greater for dry mixes than for wet ones as shown below for gravel concretes.

TABLE 30

COEFFICIENT OF THERMAL EXPANSION AS INFLUENCED BY RICHNESS OF MIX (DAVIS AND TROXELL)

Mix	Thermal Coefficient of Expansion per 1° F.	
	2-In. Slump	6-In. Slump
1:3	0.00000644	0.00000639
1:4½	0.00000600	0.00000581
1:6	0.00000580	0.00000556

The coefficient varies quite markedly with the aggregate as is indicated by Table 31.

TABLE 31

EFFECT OF CHARACTER OF AGGREGATE UPON THERMAL EXPANSION (DAVIS AND TROXELL)

Kind of Aggregate	Coeff. of Exp. per 1° F.
Quartz	0.0000066
Sandstone	0.0000065
Gravel	0.0000060
Granite	0.0000053
Basalt	0.0000048
Limestone	0.0000038

For the specimens stored at 70° F. the coefficient at 23 weeks averaged 3.5 per cent greater than the coefficient at 9 weeks.

The 100 cycles of alternating temperatures decreased the coefficient by 14.1 per cent from that which would have existed had the specimens been stored at 70° F.

The coefficient of thermal expansion appears to be fairly constant for ordinary ranges of temperature.

A fourth series of tests were conducted using high alumina cement and gravel aggregate in the proportions of 1:4.1, 1:5.1 and 1:7.6 by weight. The mixes had a slump of 3 to 4 in. and the specimens were stored in dry air continuously. The coefficients determined over a temperature range from 39° F. to 133° F. at

the age of about 4 months, shown in Table 32, indicate how little influence has either the cement ratio or temperature.

TABLE 32

EFFECT OF TEMPERATURE AND RICHNESS OF MIX ON THERMAL EXPANSION OF ALUMINA CEMENT CONCRETE (DAVIS AND TROXELL)

Mix	Coefficient of Exp. per 1° F.			
	39° to 72° F.	72° to 103° F.	103° to 133° F.	Average
1:4.1	0.00000513	0.00000548	0.00000552	0.00000538
1:5.1	0.00000489	0.00000528	0.00000502	0.00000506
1:7.6	0.00000510	0.00000519	0.00000520	0.00000516
				0.00000520

The most important work on the effect of high temperatures upon the coefficient of thermal expansion of concrete was done by Norton[12] at the Massachusetts Institute of Technology in 1910 on 6-in. and 10-in. cubes of 1:2:5 crushed stone concrete. These cubes were heated in an electric furnace until they had come completely to equilibrium at which time the changes were measured. The average results are given in Table 33.

TABLE 33

THERMAL EXPANSION AT HIGH TEMPERATURES (NORTON)

Temperature, °F.	Mean Coeff. per 1° F.
72 to 360	0.0000045 to 0.0000060
72 to 750	0.0000050 to 0.0000060
72 to 1090	0.0000045 to 0.0000050
72 to 1600	0.0000035 to 0.0000042

The values obtained at low temperature agree very well with the commonly accepted value of 0.0000055 per degree Fahrenheit. Apparently the coefficient increased slightly up to 575° F., but became smaller at higher temperatures. At about 1500° F. the coefficient of expansion became zero and above that temperature took on a negative value. Further, the blocks heated to 1500° F. did not return to their original dimensions on cooling but retained an elongation of about 75 per cent of their maximum elongation. On heating these blocks a second time no sensible permanent elongation was evident.

The effect of freezing neat cement and 1:3 mortar bars while wet has been investigated by White.[21] As water expands about 8.5 per cent while freezing, it was thought that some expansion might result from the freezing of these bars, all of which were from 3 to 6 years old. On the contrary, all of the bars contracted while freezing and at about the same rate as for higher and lower temperature ranges. After four successive freezings and thawings, the bars were entirely unaffected in length.

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(A bibliography included in number 21 above lists several foreign and early articles many of which are not included in the above bibliography. However, an attempt has been made to include those of greater interest in the present review.)

Readers are referred to the JOURNAL *of June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Specification for Supplying, Fabricating and Setting Reinforcing Steel on Ordinary Buildings

Proposed American Concrete Institute Specification No. 503
(To be Used on Buildings Requiring Considerable Steel)

BY WILLIAM F. ZABRISKIE,* AUTHOR-CHAIRMAN
Committee 503—Fabricating and Setting Reinforcing Steel.

To conform to the ideas of the Advisory Committee to make Specification 503 a companion document to Specification 502 (November JOURNAL*) and to meet the plan of the Program Committee to make them available for discussion at the 26th annual convention, Committee 503 has done a great deal of work in a comparatively short time. The specification in original draft was submitted by the Author-Chairman, Mr. Zabriskie to the critic members, Messrs. R. L. Bertin, R . W. Johnson, A. R. Lord, W. S. McKenzie and W. S. Thomson. Messrs. Lord and Thomson working together in Chicago, Mr. McKenzie in touch with Mr. Zabriskie in Detroit and all in correspondence, effected a "meeting of minds" except in a very few details. The telegraph and long distance telephone brought the document to the stage of cold type, ready for pages at the printers. The last word before press time indicated, to the best of the Secretary's knowledge and belief, a full agreement except possibly in three details in which revisions were suggested by Mr. Bertin.† The whole matter will be open for discussion at the convention (and by letter until May 1) and this discussion will be recorded in this* JOURNAL *in June 1930.*—EDITOR.

*Vice-President, General Manager Gabriel Steel Company, Detroit, Michigan.

† Mr. Bertin cited non-conformity of provision for embedment of straight bars (Sec. 5a), with Sec. 902 (b) and 903 (b) of A. C. I. Tentative Building Regulations, and a "too restrictive limitation" in "(b) Bent bars"; he objected to provisions for "Spacers and Chairs" and in "5—Bar Lists and Bending Details". He offers general objection to encroachments on design considerations. There will be opportunity to record these ideas more fully in discussion.—EDITOR

TO THE USER

THIS specification is intended for the use of Architects and Engineers, to be incorporated as a whole in the specification for particular structures by reference at the proper place in the typewritten pages and to become a part of the contract in the usual manner. It is a companion piece to American Concrete Institute Specification No. 502, Concrete Work on Ordinary Buildings, and the two specifications used together in accordance with the wording suggested on Page 3 of the November JOURNAL OF THE AMERICAN CONCRETE INSTITUTE will cover the entire concrete work on ordinary buildings. This wording is repeated in the second paragraph below.

In the preparation of this specification, the author has had the active criticism and co-operation of the critic members of the committee: W. S. McKenzie, Chief Engineer, Smith, Hinchman and Grylls, Detroit; W. S. Thomson, Chief Engineer, Kalman Steel Co., Chicago; R. W. Johnson, Engineer, Concrete Reinforcing Steel Institute, Chicago; R. L. Bertin, Chief Engineer, White Construction Co., New York; and Arthur R. Lord, President, Lord and Holinger, Engineers, Chicago.

The following wording is suggested to be used in the Architect's and Engineer's typewritten specification for individual building:

Specification for Concrete Work
Name and Location of the Building
Name and Address of Owner
Name and Address of Architect or Engineer

All plain and reinforced concrete work on this structure shall be done in strict accordance with Specifications Nos. 502 and 503 of the American Concrete Institute, which are hereby made a part of these specifications and which will be made a part of the contract for this work. All the provisions of these two specifications shall be in full force and effect except as expressly modified or supplemented in the following special instructions for this structure.

For this structure, the reinforcing steel furnished by this Contractor shall be Intermediate Grade New Billet Steel, Rail Steel (Strike out one) in accordance with Article 2 below.

(Here incorporate any necessary clauses arising from unusual conditions surrounding the work to be done, as for example: quicksand or unusual water conditions; protection of existing construction; materials purchased or furnished by the Owner direct; early strength concrete; unusual chemical exposures to be safeguarded; maintenance of manufacturing operations during construction; concrete piles, caissons or tunnels; very deep basements; under-

pinning work; special grading or maximum size of aggregates with revised fineness modulus; admixtures; etc.; etc.)

Wherever the words "Architect" or "Engineer" are used, they shall be construed to mean the Architect Engineer (Strike one out) only or his duly accredited representative on the work.

1. GENERAL

This Contractor shall furnish and place all straight and bent bars, stirrups, column ties, spirals, hanger bars, welded wire mesh, etc., required by the reinforced concrete work for this building or shown on the plans, together with all necessary inserts, spacing devices and chairs, all properly tagged with zinc marking tags and all as specified hereinafter.

2. MATERIAL

Metal reinforcement in general shall conform to the requirements of the "Standard Specifications for Billet Steel Reinforcement Bars" of Intermediate Grade (Serial Designation: A 15-14), or for "Rail Steel Concrete Reinforcement Bars" (Serial Designation: A 16-14) of the American Society for Testing Materials. The provision in these specifications for machining deformed bars before testing shall be eliminated.

Wire for welded mesh concrete reinforcement shall conform to the requirements of the "Standard Specifications for Cold Drawn Steel Wire for Concrete Reinforcement" (Serial Designation: A 82-27) of the American Society for Testing Materials.

3. PLACING DRAWINGS

This Contractor shall furnish complete erection drawings showing the location, position and identification mark of every reinforcing member, spacer and chair required for the concrete work on this building, except as it may be specified elsewhere herein that such drawings will be furnished by the architect or engineer. These drawings shall afford the steel setter complete guidance and leave no doubts or discretion to him. They shall be drawn to a scale sufficiently large to make them easily read and on a size of sheet sufficiently small for easy handling on the job under ordinary weather conditions. Where the reinforcing steel is to be laid in a certain sequence of operations, as in flat slab construction, this sequence shall be plainly noted on these drawings.

4. BAR LISTS AND BENDING DETAILS

This Contractor shall furnish the job with complete sets of bar and spiral lists and of bar, stirrup, tie, etc., bending details, as used in cutting and bending the steel in the shop. These lists and details shall show identification marks for the material corresponding with the marks on the placing drawings and with the zinc tags on the material itself. Plate I is a suggested form for such lists.

5. GENERAL DETAILS

Placing drawings and bar bending details shall follow the dimensions, notes, etc., on the structural drawings as prepared by the architect or engineer. *In the absence of such information,* the following rules shall be observed in detailing the reinforcement for this work.

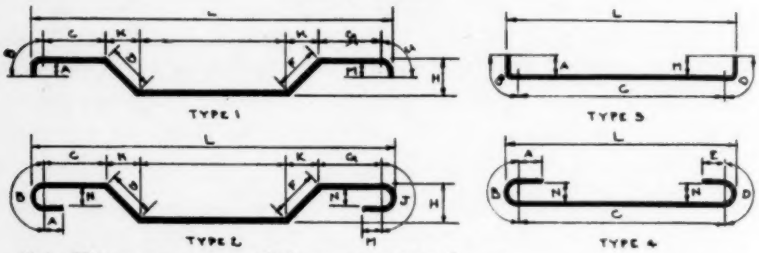
(a) *Straight Bars in Beams and Slabs*—Straight longitudinal bars used in beams, joists, ribs, slabs, etc., shall extend six inches into the support, but need not extend beyond the center of the support.

(b) *Bent Bars in Beams and Slabs*—Truss bars in continuous construction of approximately equal spans and carrying uniformly distributed loads, shall extend into the adjacent span to a point 0.225 of the clear span beyond the face of the support. On non-continuous ends, truss bars shall extend to within three inches of the outer face of the concrete.

Where the ratio of over-all depth to clear span is not greater than one to ten, in continuous beams and slabs of approximately equal spans and carrying uniformly distributed loads, the lower points of bend of truss bars shall be located at points approximately one-fifth of the clear span from the face of the support and the angle of bend shall be approximately forty-five degrees.

(c) *Stirrups*—The out-to-out width of stirrups shall be three inches less than the width of the beam, if the beam is ten inches or more in width. The out-to-out width of loose stirrups shall be two inches less than the width of the beam, if the beam is less than ten inches wide. The out-to-out width of loose stirrups in the ribs of ribbed floor construction shall be one and one-half inches less than the least width of the rib.

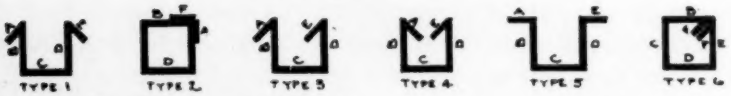
BENT BARS



Note: Tolerance on overall dimension "L" = +0" to −1"
Dimensions omitted are Zero.

No.	Size	Length	Mark	Weight	Type	A	B	C	D	E	F	G	J	M	H	K	N	L

TIES & STIRRUPS



All dimensions are out to out.
Dimensions omitted are Zero.
Tolerance on out to out dimensions = +0" to −½"

No.	Size	Length	Mark	Weight	Type	A	B	C	D	E	F	G	H	K

□ indicates square. ∅ indicates round.

PLATE 1

(d) *Column Bars*—Column vertical bars shall extend from floor to floor plus a lap of twenty-four bar diameters, but not less than eighteen inches, for deformed bars; or plus a lap of thirty diameters, but not less than twenty-four inches for plain bars. Column bars for the top story shall stop at a point three inches below the top of roof slab.

Column bars shall be detailed as bent bars on all faces where the face of the next column above in the same tier is offset two inches or more from the corresponding face of the column section being considered.

(e) *Column Ties*—The out-to-out dimensions of column ties shall be four inches less than the corresponding outside dimensions of the column.

(f) *Column Spirals*—Where less than four beams frame into a column at its top the spiral shall extend from the top of the lower floor slab to within three inches of the bottom of the floor slab above. Where four beams frame into a column at its top the spiral shall extend from the top of the lower slab to the underside of the shallowest of the four beams.

In columns supporting flat slabs the spiral shall extend from the top of the lower floor slab to the underside of the drop panel above, or the underside of the floor slab above if no drop panel is used.

The out-to-out diameter of spirals is to be three inches less than the outside dimension of columns except in the cases of columns exposed to the weather, in which case the spirals shall be four inches less than the outside diameter of the column.

Column spirals shall be provided with not less than three spacers which shall be located at even intervals when the spiral is placed in the form.

(g) *Dowels*—Dowels shall be at least forty-eight bar diameters but not less than thirty-six inches in length for deformed bars; or at least sixty bar diameters, but not less than forty-eight inches in length for plain bars.

(h) *Footing Bars*—Footing bars shall extend to within three inches of the sides of the footing.

(i) *Pins and Sleeves on Bending Equipment*—For ordinary bending of slab or beam steel, the diameter of pin used shall be not less than four times the diameter of the material being fabricated. No pin smaller than three inches in diameter shall be used for this purpose.

Semi-circular hooks shall be formed around a sleeve not less than six times the diameter of the bar in the case of intermediate grade billet steel, and eight times the diameter of the bar in the case of rail steel.

Right angle bends in column ties and U-stirrups, not larger than one-half inch in diameter, and hooks at end of same, shall be formed around a pin not less than one and one-quarter inches in diameter, or around a die giving an equivalent curvature.

(k) *System of Marking*—The marking of reinforcing steel shall be done by zinc tags with embossed letters conforming to the standard tag-marking wheels, as given by the Code of Standard Practice of the Concrete reinforcing Steel Institute.

6. BUNDLING AND TAGGING

The reinforcing steel for this work shall be shipped to the job in standard bundles and tagged with embossed zinc shipping and marking tags, prepared in accordance with the Code of Standard Practice of the Concrete Reinforcing Steel Institute, (Table 1).

7. SHIPMENTS, HANDLING AND STORAGE

Where storage space at the job is limited, shipments shall be made so as to avoid confusion. Bundling and tagging shall be arranged to permit of the shipment of a portion of a floor only, if required, in accordance with a schedule to be agreed upon between the job and the shop.

Reinforcing Steel shall be stored on the job in separate piles or racks and shall be kept separated at all times so as to avoid confusion or loss of identification after bundles are broken open.

8. SPACERS AND CHAIRS

Spacing devices, chairs, etc., shall be furnished and installed in accordance with the Code of Standard Practice of the Concrete Reinforcing Steel Institute, (Plate 2).

9. FIELD PLACING

All steel shall be placed in strict conformity with the requirements of the engineering design drawings, both as to location,

TABLE I

Note—The following rules apply to carload and less than carload lots, except where differences is specifically set forth.

	Straight Bars	*Bent Bars*
(a) Weight of bundle:	Bundles limited to one size and one length not to exceed 150 lbs.	150 Lbs.
(b) Gauge of wrapping wire to be used:	No. 12 or heavier.	No. 12 or heavier.
(c) Wrapping wires per bundle:	One wire every ten feet or fraction thereof with a minimum of two wires.	Not less than requirements for straight bars.
(d) Tag to be made of:	Linen or rope for address.	Linen or rope for address. Zinc for identification.
(e) Number of tags on each bundle:	One address tag, one identification tag.	One address tag, at least two indentification tags.
(f) Information to be put on tag:	Name of customer or order number. Number of pieces, size, length, mark if any. On less than carload lots the following should appear: Name and address of customer. Number pieces, size, length and mark if any.	Linen or rope tag: Customer's name or order number. Zinc tag: Mark. On less than carload lots customer's name and address must appear on linen or rope tag and mark on zinc tag.
(g) Information to be affixed to tag by:	On linen or rope tag: Tag addressing machine or Higgins waterproof ink. On Zinc tag: Stencil press or embossing machine.	Same as straight bars.
(h) Tags attached to bundles by:	Linen or rope tag to be attached by running bundling wire through eyelet before twisting. Zinc tag to be tied to bar with No. 18 wire.	Same as straight bars.

position and spacing of members, and also the thickness of concrete protection afforded.

All steel shall be supported and secured against displacement by the use of adequate and proper supporting and spacing devices, tie wires, etc., so that it will remain in its correct location in the finished structure.

10. Re-bending and cutting

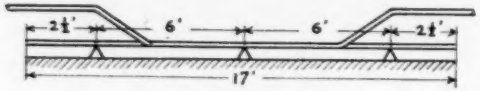
Re-bending of reinforcing steel on the job to make it fit the existing conditions shall only be done with the approval and under

PLATE 2

SLABS

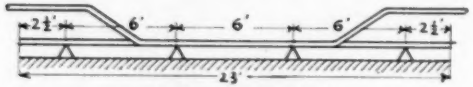
Ribbed Floor Slabs

One Way Slabs



BEAMS (*ORDINARY*)

Single Layer



BEAMS (*HEAVY*)

Three Layer

Bars shall be Accurately Located and Firmly Held—All reinforcing steel shall be accurately located in the forms and firmly held in place before and during the pouring of concrete, by means of metallic supports, spacer bars, wires or other devices adequate to insure against displacement during the course of construction, and to keep the steel at the proper distance from the forms.

Bar supports and spacers are to be sufficient in number, and sufficiently heavy to properly carry the steel they support. The number shall be such as to give support not less than the following:

SLABS

Rib Floor Slabs	
Clear Spans	Rib Chairs per Rib
Over 0 ft. to 10 ft.	2
Over 10 ft. to 20 ft.	3
Over 20 ft to 30 ft.	4

To support ends of bent bars in ribs—use a ⅜" round bar at each side and parallel to the supporting beam or wall, such ⅜" round bar to be held above the permanent or removable tile.

Solid Slabs with Principal Reinforcing in One or Two Directions	
Clear Spans	Supporting Spacers per Panel (One Direction Only)
Over 0 ft. to 7 ft.	2 lines
Over 7 ft. to 17 ft.	3 lines
Over 17 ft. to 27 ft.	4 lines

To support ends of bent bars in slabs—use bar not less than five-eighths inch (⅝") diameter carried on high chairs spaced not more than four feet (4') apart. These bars are to be used on two opposite sides of panels when principal reinforcing is in one direction only, and on all four sides of panels when principal reinforcing is in two directions.

BEAMS

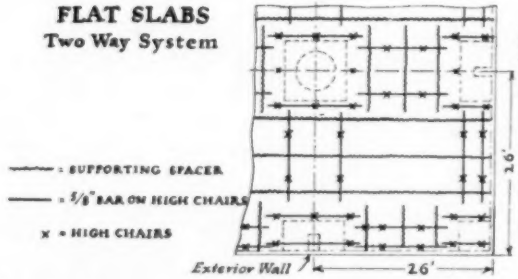
Ordinary Beams (Bars One (1) Inch Square and Smaller)

Clear Spans	Number of Beam Chairs					
	Single Layer of Bars	Two Layers		Three Layers		
		Lower	Top	Lower	Middle	Top
Over 0 ft. to 14 ft....	2	2	2	2	2	2
Over 14 ft. to 23 ft...	4	3	2	3	2	2
Over 23 ft.....	See table below					

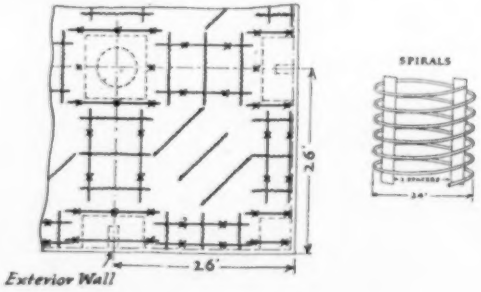
PLATE 2 (CONTINUED)

FLAT SLABS

Two Way System



Four Way System



Beams or Slabs with Large Number of 1⅛ or 1¼ Inch Bars

Clear Spans	Number of Beam Chairs					
	Single Layer of Bars	Two Layers		Three Layers		
		Lower	Top	Lower	Middle	Top
Over 0 ft. to 15 ft.	2	3	2	3	2	2
Over 15 ft. to 30 ft.	4	5	2	6	2	2
Over 30 ft. to 45 ft.	6	7	3	9	3	3
Over 45 ft. to 60 ft.	8	9	3	11	3	3
Spacing of Beam Chairs in spans other than above.						
All Spans	8'0"	7'0"	14'0"	5'0"	10'0"	10'0"

FLAT SLABS—*Two and Four Way Flat Slabs*

Spans (center to center of columns)	Supporting Spacers		High Chairs (to support ⅝ inch bar under ends of bent bars)	
	Column Strip or Direct Band	Middle Strip or Diagonal Band Bottom Layer	Support for Column Head Reinforcement	Support for Negative Reinforcement Middle Strip
Over 0 ft. to 18 ft.	3	2		
Over 18 ft. to 26 ft.	3	3		
Over 26 ft. to 36 ft.	4	4		
Around Interior Columns			8	
Around Exterior Columns			5	
Around Corner Columns			4	
In Interior Panels				8
In Exterior Panels				10
In Corner Panels				12

In roof slabs use one (1) more supporting spacer under column strips, direct bands, and bottom layers of middle strips or diagonal bands and one more ⅝ inch chair bar at column heads, than the number shown in the table above.

SPIRALS—*Columns with Spiral Hooping*

Core Diameter (Outside to Outside)	Number of Spacers
Over 0 in. to 24 in.	2
Over 24 inches	3

the personal direction of the Architect or Engineer. Bars shall not be shortened or hooks cut off on the job, except under the approval and direction of the Architect or Engineer.

11. PICK-UP MAN

A capable steel setter shall be present on the work at all times during the placing of concrete, to look after the reinforcing steel and to re-set properly any steel displaced by runways, workmen, or other causes.

See appendix, "Steel Setters' Primer" p. 455.

Readers are referred to the JOURNAL *for June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Steel Setters' Primer

Appendix, Report Committee 503, Fabricating and Setting Reinforcing Steel

BY WILLIAM F. ZABRISKIE,* AUTHOR-CHAIRMAN

This committee has done not only what it was asked to do but its Author-chairman, Mr. Zabriskie, feeling that the fundamentals of steel reinforcing should be "got across" to the steel setter, has appended a further document: a "Steel Setters' Primer." A steel setter is a very necessary individual in whom a knowledge of reinforced concrete design is not considered necessary. Yet the integrity of the reinforced concrete depends not infrequently on the importance which he attaches to the accurate placing of steel. Mr. Zabriskie asks him to "picture where the member would be likely to break if no steel were provided" and helps him to conjure up the picture.—Editor

Concrete is one of the most adaptable building materials we have to work with today. In addition to its great strength as a masonry material for walls and foundations, it can be used, when reinforced, for floors, roofs and other building members by the addition of reinforcing steel. The one property of plain concrete which limits its use is its low resistance to tensile or pulling stress in which property it is very much inferior to timber or steel. In general, reinforcing steel is supplied to overcome this deficiency. Except in those cases where reinforcing is used to assist the concrete in carrying compression or pushing stresses, such as in columns or occasionally in other members, the steel setter will be guided by picturing where the member would be likely to break if no steel were provided. In simple members, experience with other materials will serve as a guide. At other points, the

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stresses may not be so fully understood and in order that the steel setter may have a better understanding of what duty the steel is called upon to fulfill, the following brief description is given of the more general units of construction in which reinforcing steel is required.

FOOTINGS

Reinforced concrete footings are generally of two types, spread footings and combined or beam footings. Spread footings are generally square and the steel usually occurs near the bottom of the lower block of the footing. It is generally installed in two layers at right angles to one another and raised far enough above the bottom of the excavation to permit complete embedment in the concrete. The purpose of a concrete footing is to distribute the load from the column to an area of ground sufficiently large so that it can safely sustain the load. The upward pressure of the soil against the bottom of the concrete footing has a tendency to cause the lower block of concrete to break off at approximately the edge of the upper block or the edge of the column. This tendency of the concrete to break off is resisted by reinforcing steel placed near the bottom of the footing as previously described.

In the case of a combined or beam footing, two or more columns are placed on a single footing which distributes the loads of all columns to the ground. In order to spread the load from the columns to the ground, the footing must act as a beam of sufficient strength so that the load applied to the soil will be as great between the columns as it is under them. In this type, the principal steel is generally in the top of the footing between the columns and in the bottom of the footing under the columns. In wide footings there is frequently required steel at right angles to the length of the footing, placed in the bottom and near the columns to distribute the load across the footing and to prevent it from splitting longitudinally. Bent bars and stirrups are also frequently called for in beam footings to prevent the formation of cracks due to diagonal tension. This will be discussed more fully under the general subject of beams.

In footings carrying reinforced concrete columns, dowel bars are usually provided. These bars form a splice with the column verticals and serve to transfer the load in the verticals to the

footing by means of bond with the concrete. In the case of square spread footings with two steps, these dowels are generally placed in the upper block only, but where the bars are large and the upper block is shallow, it may be necessary to extend them into the lower block. In setting a dowel, it should be remembered that there should be practically the same length of dowel imbedded in the concrete below the joint as there is projecting above the joint. Dowels should be carefully set to correspond with the location of bars in the column to be carried by the footing. Although it is frequently possible to set dowels after the footing concrete is placed, it is very bad practice to wait until the concrete has hardened so that it is necessary to drive them in with a sledge as is sometimes done. This practice not only greatly weakens the bond of the concrete to the steel dowel, but is likely to rupture the partially hardened concrete and seriously weaken the whole footing.

COLUMNS

Reinforced concrete columns are built of two classes, tied columns and spiral columns. In the first class, columns may be built of any shape, but are generally square or rectangular. In the case of square or rectangular columns of moderate size, they are usually reinforced by vertical bars placed near each corner of the column and supported by ties, one-quarter inch round or larger, spaced about twelve inches apart vertically. The purpose of the verticals is to assist the concrete in carrying the load on the column and to prevent cracks from forming in those cases where there is a tendency for the column to bend. The purpose of the ties is to help in holding the vertical bars accurately in position and also to prevent outward buckling of these bars under the column load.

Columns of the spiral class are reinforced by a continuous spirally wound hoop extending from the top to the bottom of the column, and, in addition, vertical rods are placed inside of this hoop. This type of column is generally used where heavy loads are to be carried, as the presence of the spiral makes it possible to use a smaller column than in the case of the type previously described. The purpose of the spiral hooping is to prevent a splitting of the column when under heavy load.

If you were to step on the upper end of a paper sack of flour or cement, you would find that the sack would split down one side, permitting the flour or cement to run out and the sack to collapse. This same tendency is true in a concrete column when heavily loaded, and it is the spiral hooping that resists this force and prevents the splitting of the column. For this reason, it is essential that the hooping be continuous from end to end. If it is cut by electricians or plumbers or any other tradesmen, the spirals should be replaced or repaired by welding or splicing in additional hoops. The vertical bars in this type of column assist the concrete in carrying the vertical load just as they do in the tied column.

In all concrete columns, it is very important that the steel be located and held accurately at the proper distance from the forms. This is particularly important in the case of exterior columns when the outer face is to be left exposed to the weather in the finished building. Even though precautions are taken to make good concrete, moisture has a tendency to work back from the face of exposed concrete. When this moisture reaches the steel, rusting occurs, and the rust, forming a scale on the steel, creates a pressure from inside of the concrete which cracks and spalls the surface, permitting more moisture to enter. This is one of the objections raised to the use of reinforced concrete in exposed locations, but experience teaches us that it is an objection that may be readily overcome through the application of care and intelligence in the placing of the steel in combination with equally careful treatment of the concrete.

BEAMS

Reinforced concrete beams are used to span between walls, piers or columns for the support of concrete slabs or other loads. To understand the necessity for reinforcing beams, it is desirable to consider how a timber beam would fail under load. Experience will show that such a beam will fail by cracking or parting the fibers near the center of the span or at the bottom of the beam. Since concrete is weak in resisting tensile or pulling stresses, it is necessary to have steel to resist these stresses. The principal reinforcing of a concrete beam consists of bars placed in the lower portion of the beam and running throughout its length

to resist these pulling stresses. This steel should be placed somewhat above the bottom of the beam so that the concrete by bonding to the bar will transmit the tensile or pulling stresses to it. The concrete under the bars is also of importance in securing proper protection for the bars against fire and corrosion. The following rules will be found useful in locating and spacing this steel where a large number of bars are called for.

No. 1—Main bottom steel should be not less than one and one-half inches from the bottom or sides of the beam boxes to secure adequate fire and rust protection.

No. 2—Bars should be placed so that the minimum distance center-to-center between parallel bars will be not less than two and one-half times the diameter for round bars, or three times the side dimension for square bars.

No. 3—In case hooks are provided at the ends of the bars to aid in anchoring them to the concrete columns, these spacings from center-to-center of bars may be made two diameters for round bars, or two and one-half times the side dimension for square bars, but in no case should the clear spacing between bars be less than one inch.

In order to secure the proper spacing and location of bottom steel, an adequate number of beam chairs should be used. Where it is necessary to place the steel in more than one layer, a second set of chairs should be used on top of the first layer of bars, and the same rules for spacing should be applied to the upper layer as was given for the lower layer.

Where the plans require several beam spans supported by piers or columns in a row, it is necessary and customary to provide steel in the top of the beam over the supports. To understand the necessity for this steel, you must remember that when the beam is loaded the column or pier exerts a pressure upward on the beam at this point and that the beam tends slightly to bend or sag between the columns or piers. If you will make a model by taking a light piece of wood, such as a yard stick or lath, laying it across several supports and loading it between them, you will see that the stick is curved over each of these supports. A beam continuous over supports is similarly curved and due to this curvature over the supports cracks will form at these points unless reinforcing steel is supplied at the top of the beam. The use of

this steel is very important since if cracks are permitted to form they will sometimes extend from the top of the beam through to the bottom. If this crack occurs at or near the edge of a column or pier it will be seen that the beam can no longer carry load to its support. This top steel should be accurately placed and spaced in accordance with the same rules as given before for bottom steel, and with a protection of concrete over the bars at the top of not less than one inch.

Where a concrete beam is carried by a concrete column with no beam on the opposite side of the column it is frequently considered desirable to place hooks at the end of the bars both in the top and bottom of the beam to prevent these bars from slipping or pulling out of the concrete column. The hooked ends of these bars should be extended into the column as far as possible considering the length of the bar and the location of other bends, but should never be nearer the surface of the concrete than one and one-half inches.

Except in small and unimportant beams, it is generally desirable and necessary to add additional steel near the ends of the beams to aid in preventing the beams from cracking near the supports. This is spoken of as "shear reinforcing." When a reinforced concrete beam is heavily loaded, in addition to there being a tendency for it to crack at the bottom near the center and at the top over the supports, there is also a tendency for cracks to form near the supporting column or pier. These cracks run at an angle from the bottom of the beam near the support upward and toward the middle of the span. They are caused by a tendency of the center portion of the beam to drop away from that part which is over the column or pier. This shear reinforcing is supplied in several forms; its most general form is that of U or double-U stirrups placed in the beam boxes previous to placing the main beam steel. It is also customary in beams to supply shear steel partly or entirely in the form of bent bars. This form of shear reinforcement is generally supplied by bending up at an angle some of the main bars from the bottom of the beam to a point near the top of the beam, then flattening them out and extending them across the column or pier for use as top steel. The bend of these bars generally makes an angle with the bottom of the beam of between thirty and sixty degrees. As before explained, U-

stirrups are placed in the beam boxes before the main steel is set. After the main steel is placed, the stirrups should be wired securely to the main bars so as to be directly in contact with the bottom of them. In the case of U-stirrups having a hook provided at their upper ends, it is very desirable to provide a spacing bar running through and wired to the inside of the hooks. Where top steel is provided for beams over supports, it is frequently possible to securely wire the sides of the stirrups to the top steel. It is very important that all stirrups be kept away from the bottoms and sides of beam boxes a sufficient distance to secure a protection of concrete over the steel. For beams ten inches wide or wider, stirrups should have one and one-half inches of concrete between the side of the stirrup and the beam form. In the case of beams narrower than ten inches, this protection may be reduced to a minimum of one inch.

SLABS

Reinforced concrete slabs are generally used for the floors and roofs of buildings. They may be supported by masonry walls or by beams of steel or reinforced concrete. A number of different designs of slabs are in general use. All of these are based on the same general principles but are sufficiently unlike so that each will be discussed under a separate heading.

Solid Slabs—A solid slab is the simplest form of reinforced concrete slab. It consists merely of concrete poured on a solid centering and reinforced by bars spaced at regular intervals. As explained previously, the bars are intended to prevent the cracking of the slab and are used at a point where the slab would tend to break due to tensile or pulling forces. Experience with other building materials, such as wooden joists, will show you that when loaded they generally fail by breaking of fibers at the bottom of the joists. Thus, in slabs spanning between supports there is always a tendency for the slab to fail by tension in the bottom. Reinforcing steel, therefore, is provided running between the supports and regularly spaced in the bottom of the slab. Experience has shown that unless some steel is placed at right angles to this principal steel there is a tendency for slabs to crack parallel to the direction of the bars. It is therefore customary to provide smaller bars at right angles to the main bars, generally spaced

twelve or sixteen inches on centers. This steel is frequently referred to as temperature reinforcement because the cracking is considered to be caused by expansion and contraction of the slab due to changes in temperature. In placing slab steel, it is important to see that the bars are properly spaced, are parallel with one another, and are properly supported a sufficient distance above the form to secure imbedment in the concrete and to protect them from the effects of a fire under the slab. Most specifications require a minimum protection of three-quarters of an inch under the bars.

Where the plans require several spans of slab supported by parallel walls or beams, it is customary to provide steel in the top of the slab over the supports. The purpose of this steel is to prevent cracks forming over the beams, as described previously under the heading of BEAMS. The use of this steel is very important since if the cracks are permitted to form they will sometimes extend through the slab which is then cracked away from its support and can no longer safely support any load. This top steel should be placed as near the top of the slab as possible, generally being imbedded a distance of three-quarters of an inch from the top of a solid slab. In the case of slabs having equal spans, it should extend to approximately the one-quarter point of the clear span in the top of the slab.

Terra Cotta Tile and Concrete Joist Slabs—These are constructed sometimes on solid wood centering and sometimes on open centering consisting of 2 x 8 or 2 x 10 inch planks laid on 4 x 4's and spaced approximately sixteen inches on centers. On these are laid hollow terra cotta tile, end to end, so that the side of the tiles and the two-inch plank constitute a trough in which to place the concrete of the joist. After setting the reinforcing steel, the concrete is placed between the tile and is carried sometimes to the top of the tile, but more frequently to a point two inches above the tile. This type of construction can be used economically on much longer spans than can a solid slab. The reason for this is that for any given total depth of slab this construction weighs much less than a solid concrete slab since the tile, being hollow, weigh much less than the concrete which they replace. This construction also has the advantages of insulating against heat or cold due to the air space in the tile and also fur-

nishes a better base for plastering than a plain concrete slab due to the scoring on the underside of the tile. Reinforcing in this form of construction consists of one or two bars placed in the bottom of each joist and supported above the bottom of the joist and spaced between the sides of the tile. Top steel should be provided over walls or beams in the same way and for the same reasons as were discussed under beams and solid slabs.

Ribbed Slabs—Ribbed slabs are those in which the floor or roof consists of a series of concrete joists or ribs with a thin slab, usually two or two and one-half inches in thickness, between them. These slabs are formed on an open or solid wood centering and the spaces between the ribs are formed either of corrugated metal forms which remain permanently in the slab, or heavy gauge steel forms which are removed after the concrete has set. This type of floor can be used on very long spans because of its extreme lightness. It is also the most economical type of floor from the standpoint of the amount of concrete which is used. Reinforcing for this type of floor consists generally of two bars placed in the bottom of each joist or rib and properly supported above the soffit form and between the two side forms of the joists. Bars should be provided over supports as previously discussed under BEAMS and solid slabs. In addition to this reinforcing, it is customary and important to provide reinforcement in the thin top slab. This is sometimes provided by one-quarter inch bars running at right angles to the ribs and placed above the metal forms. It is also frequently provided by using a wire fabric placed across the top of the forms. Where this type of reinforcement is used, the fabric should always be placed so that the closely spaced wires run at right angles to the concrete joist. Where light bars or mesh are used to reinforce this upper slab it is important that when the concrete is placed the bars or mesh be raised slightly above the metal form to secure some imbedment.

In ribbed slabs of long span or where used for heavy loads, it is sometimes necessary to use stirrups in the ribs to prevent the formation of cracks near the supports. This is spoken of as "shear reinforcing," and acts in a joist or rib in the same way that shear reinforcing acts in beams as previously described. This "shear reinforcing" is generally supplied in the form of stirrups or U-shaped bars placed vertically in the rib before the main bars

are placed. It is sometimes supplied in the form of wire loops wrapped around the bar and extending up to the top of the slab, and it may also be supplied by cutting away a portion of the reinforcing bar and bending it upward near the support. When loose stirrups are used, it is important that they be accurately positioned according to the spacing called for and securely wired so as to remain in a vertical position during the placing of the concrete.

CANTILEVER BEAMS AND SLABS

In building construction it is sometimes necessary that a beam or a slab should project or extend beyond a support with no additional support at its outer end. This is known as "cantilever construction." Beams which project in this manner beyond walls or columns are spoken of as "Cantilever Beams," and slabs which project beyond beams or walls are spoken of as "Cantilever Slabs." By considering the similarity to a projecting wooden beam, it will be seen that the tendency for the beam to crack is at the top, this tendency being greatest near the support. In cantilever reinforced concrete construction, there is similarly a tendency to crack in the top of the beam or slab for the whole length which projects beyond the support and also for a considerable distance inside of the support, since the projecting end of the beam or slab must be balanced by an inner portion or anchor arm. In cantilever beams and slabs it is therefore necessary that the principal reinforcing steel be placed in the top and extend in the same direction as the cantilever projects. It is extremely important that this steel be held securely in its proper position since if it becomes lowered during the placing of the concrete the cantilever construction will be very greatly weakened.

In cantilever slab construction, it is important to use a sufficient number of substantial chair bars and high chairs to hold this steel in position, both in the cantilever portion and in the anchored portion of the construction.

FLAT SLABS

The term "flat slab" is generally applied to a floor or roof slab constructed as a solid slab without beams with the principal reinforcing running in two or more directions. In this type of construction, the interior columns are generally supplied with

enlarged column capitals, and the exterior columns are supplied with either a half capital or a bracket. Thickened portions of the slab, known as dropped panels, are generally provided at interior columns and sometimes at exterior columns in this form of construction. In flat slab construction, the reinforcing may be of several types and the steel setter must be guided by the information given on the erection drawings. The reinforcing steel is generally called for in "bands." The steel in these bands is placed in much the same way as reinforcing steel for any solid slab.

While it is not considered possible to give any particular rules for the placing of flat slab steel, it may be desirable to say a word of caution regarding certain points in this form of construction:

1. The heaviest stresses occurring in a flat slab are those in the top of the slab around the column head. At this point the steel is always provided in the top of the slab. It is very important that this steel be properly placed at the right elevation, and that sufficient high chairs and chair bars be provided to insure its remaining at the right level in the finished construction.

2. In placing the various bands of steel, care should be taken that there are not more layers of steel above the column than are absolutely necessary. In a two-way slab there should never be more than two layers of steel at the column capital, and in a four-way slab there should never be more than four layers of steel at a column capital.

3. When a flat slab is supported at its edge by a wall beam, it is important to provide top steel in the slab anchored into the beam throughout its full length.

4. At exterior columns it is equally as important to see that the top steel is properly placed and supported as it is at interior columns, possibly more so, as additional heavy bars are frequently required in the top of the slab at exterior columns.

RETAINING WALLS

Reinforced concrete retaining walls are generally of two types: cantilever construction and slabs spanned between supports.

In building construction, the most frequent form of retaining wall is of the latter type. This takes the form of a reinforced concrete slab spanned either vertically between floors which act

as supports, or horizontally between columns, pilasters or cross walls which serve as supports. In either case, the principal slab reinforcing should be in the inner face of the wall or on the opposite side from which the earth pressure is applied. Where walls are continuous beyond supports, bars should be supplied in the outer face of the wall to prevent cracks at this point in the same way as top steel is provided in slabs over supports.

The cantilever type of retaining wall is frequently used for grade separations, wing walls of bridge abutments and any location where a high bank of earth must be retained by a wall without a support at the top. In this type of wall, a very wide and heavy footing or base is provided and the wall projects as a cantilever above this base. Reinforcing steel must be provided in the wall, in the base, and between the wall and base to prevent cracks forming due to the pressure of the earth tending to overturn the wall. The wall acts like a cantilever slab as previously described, projecting from the base in the same way that a slab projects beyond a beam or wall. The tendency to crack in this construction is on the side where the pressure is applied, and the reinforcing bars are therefore placed on the side of the wall next to the earth bank. These bars must be adequately anchored into the wall footing or base, either by being placed in one length before the base of the wall is poured or by being spliced onto dowels extending above the base.

In either type of retaining wall, it is important to provide adequate temperature steel at right angles to the main reinforcing and securely wired to it, and to keep the bars sufficiently far away from the forms to permit of an ample covering of concrete. This distance is generally indicated on the engineer's drawing but it should not be less than two inches.

Readers are referred to the JOURNAL *for June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Feb. 1930

Proposed Specifications for Ready-Mixed Concrete

Progress Report of Committee 504. Specifications for Centrally Mixed Concrete

MILES N. CLAIR,* AUTHOR-CHAIRMAN

Mr. Clair, while acknowledging valuable suggestions, assumes full responsibility for this report. Discussion by critic members of his committee and others may be counted on to develop the adequacy of the proposed specification for "ready-mixed" concrete or to show wherein it does not meet all the needs and conditions of this new rapid development in the concrete field. With all the discussion in hand and with the information being prepared by Committee 602 on design and operation of central mixing plants a specification can doubtless be drawn for early tentative adoption. This report is on the 26th Annual Convention program.—EDITOR

INTRODUCTORY

Although designated as a committee to develop a specification for "Centrally Mixed Concrete," it is understood that the entire field of "ready-mixed-to-use-as-received-on-the-job concrete" is to be considered. There is presented in this "progress" report a proposed tentative specification for ready-mixed concrete, suitable for use by architects and engineers, which is followed by some discussion of the items where deviations from the usual standard specifications occur.

The author acknowledges with appreciation the criticism and cooperation of the critic members of the committee,† as well as

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†Alexander Foster, P. J. Freeman, Frank I. Ginsberg, R. S. Greenman, W. E. Hart, Manton E. Hibbs, C. E. Nichols, T. C. Powers, H. F. Thomson.

the valuable suggestions received from other members of the society, *previous to the writing of this report.* It is hoped that the society in general, as well as those particularly interested in this relatively new development in the field of concrete construction, will offer further constructive criticism.

PROPOSED SPECIFICATIONS FOR READY MIXED CONCRETE

1—GENERAL.

(a) These specifications cover the conditions affecting the use of Ready-Mixed Concrete.

(b) The term "Ready-Mixed" is used in these specifications as including commercially-produced concrete, whether made in a Central Mixing Plant and conveyed to the place of deposit, or whether the materials are measured at a Central Proportioning Plant, and the mixing done in transit or at the place of deposit.

(c) The place where the materials are measured is referred to in these specifications, as "the plant."

2—MATERIALS.

(a) Portland cement shall conform to the Standard Specifications and tests for portland cement (C9-26) of the American Society for Testing Materials.

(b) Concrete aggregates shall conform to the Tentative Specifications for Concrete Aggregates (C 33-28T) of the American Society for Testing Materials.

(c) Water used in mixing concrete shall be free of injurious substances.

(d) Cement and aggregates shall be stored at the plant in a manner to prevent deterioration or the intrusion of deleterious matter. Any material which has deteriorated or has been damaged shall be promptly and completely removed from the plant.

(e) No change shall be made in the materials used, during the course of the work, except with the permission of the Architect or Engineer.

(f) Facilities shall be provided at the plant for the inspection and test of materials by the Architect or Engineer.

3—CONCRETE QUALITY AND PROPORTIONING

(a) The working stresses for the design of the structure are based upon the minimum ultimate 28 day strength of the con-

crete to be used. The strength of the concrete, for which all parts of the structure were designed, is shown on the several plans.

(b) The concrete shall be so proportioned and mixed that when delivered to the forms it will have the consistency specified and will develop the strength as specified and shown on the several plans, at 28 days, as determined by the "Standard methods of making compression tests of Concrete" (C 39-27) of the American Society for Testing Materials.

(c) The method of measurement of concrete materials shall be such that the proportions can be readily adjusted to assure concrete of the quality specified. The operation of the plant shall be under the direct supervision of a competent concrete testing engineer.

(d) Each load of concrete shall be accompanied by a certified statement of the quantities of all the ingredients used and the quality of the concrete delivered. The organization supplying the ready-mixed concrete shall furnish sufficient test data certified to by a competent concrete testing engineer to enable the Architect or Engineer to determine whether the quantities used will give concrete of the desired quality.

(e) During the progress of the work, a reasonable number of compression tests shall be made as may be required by the Architect or Engineer, but at least one specimen shall be tested for each 50 cubic yards of concrete being placed. The tests shall be made in accordance with the "Standard Method of Making and Storing Specimens of Concrete in the Field" (C31-27) of the American Society for Testing Materials.

(f) Facilities shall be provided at the plant to enable the Architect or Engineer to easily check at anytime the accuracy of the measurements.

4—MIXING AND TRANSPORTING CONCRETE.

(a) Central Plant-Mixed Concrete

1—The concrete shall be properly mixed before being loaded into the transporting vehicle.

2—Transporting vehicles shall be so constructed as to insure rapid delivery of the concrete to the job without segregation or loss of the ingredients.

3—Ordinary flat bottom trucks shall not be used to transport

central plant-mixed concrete when the consistency of the concrete is wetter than a 2½ inch slump.

4—Central Plant-Mixed Concrete shall not be transported in any case for a period in excess of one hour after the addition of the cement.

(b) Central Plant-Proportioned truck-mixed concrete

1—If the aggregates and cement are introduced into the transporting and mixing vehicle together, the procedure shall be such that they are uniformly distributed throughout the batch when the loading is complete.

2—Vehicles used for transporting and mixing shall be so constructed as to insure rapid delivery and proper mixing without segregation or loss of the ingredients.

3—Central Plant-proportioned truck-mixed concrete, when the aggregates and cement are batched together, shall not be transported in any case for a period in excess of one hour after the addition of the cement to the aggregate.

4—Means shall be provided so that the quantity of water added to the mix can be readily checked.

(c) Under no circumstances shall partially-hardened or retempered concrete be placed in the structure.

(d) During weather approaching freezing temperatures, the concrete as delivered to the job, shall have a temperature not less than 50° F. and not more than 120° F.

(e) Each load of concrete shall be accompanied by a certified statement as to the time at which the cement was added.

5—DELIVERY.

(a) The organization supplying concrete shall have sufficient plant capacity and transportation apparatus to insure continuous delivery at the rate required.

(b) The interval between batches for a pour shall not exceed thirty minutes or in any case be so great as to allow the concrete in place to partially harden.

(c) The methods of delivering the concrete shall be such as will facilitate placing with the minimum of rehandling and without damage to the structure or concrete.

DISCUSSION BY AUTHOR-CHAIRMAN

Commercial ready-mixed concrete plants have been in operation for from five to eight years, and essentially similar concrete-producing and distribution systems have been employed for many years on the larger construction projects where concrete had to be deposited over a large area. The commercial concrete plant, therefore, cannot be considered entirely new either in practice or principle. The increase in the number of plants, however, has been particularly rapid within the last two years, due partly to the increased demand by engineers and architects for concrete of known quality, and partly because of the appreciation by contractors of the advantages in many cases of eliminating the job-mixing plant.

The distribution of the commercial concrete plants is now so general that practically every engineer and architect has before him, or will have in the near future, the question of the use of ready-mixed concrete. A specification is, therefore, vitally needed that will insure the engineer or architect of ready-mixed concrete of the quality specified, and at the same time, will allow the commercial concrete producer to take full advantage of the savings to be obtained from large scale production and proper use of his materials. The proposed tentative specifications, which are given in this report, were developed with these two requirements in mind and follows essentially the form and wording of the "Specifications for Plain and Reinforced Concrete" given on pages 46 to 50 inclusive of the third edition of the pamphlet entitled "Design Control of Concrete Mixtures," published by the Portland Cement Association.

The proposed tentative specifications are not intended as a general specification for plain and reinforced concrete, but rather as an addition to such specifications, when the architect or engineer has to consider the use of ready-mixed concrete on work over which he has supervision. The two systems in general use today, in the commercially-produced concrete field, are the Central Mixing Plant system and the Central Batching Plant truck-mixing system. The procedure in the Central Mixing Plant system is to proportion the materials and to mix the concrete at a central plant and to transport the mixed concrete to

the point of deposit in trucks equipped with or without some form of agitating apparatus. This system is essentially that employed in the past on large construction projects such as dams. The procedure in the central batching plant, truck-mixing system is to proportion the materials at a central plant and to transport the batched materials, to which water has not been added, to the point of deposit in special mixer trucks which mix the concrete during transportation or after arrival at the job. This system is similar to that employed in the past on large road construction projects where materials were batched at a central point and delivered to the mixer which moved along as the work progressed. There is also used, to some extent, a modification of the central-batching plant system which corresponds more closely to the methods used in road work, in which the materials are batched at a central plant, but are kept separate and are delivered to a mixer on the job. The usual form of specification should satisfactorily cover this special system, as there occurs no contact of cement, aggregates, and water, preliminary to the mixing and immediate placement of the concrete.

Proper materials must be used in order to obtain good concrete economically and, therefore, the restrictions in regard to the quality and storage of the cement, aggregates, and water must be essentially the same for ready-mixed concrete as for job-made concrete. The commercial concrete plant may have an advantage over the job plant, however, because of the relatively larger consumption of materials which should enable them to demand better materials and better prices. This buying power of the commercial concrete plant, and the desire to increase income may lead in some cases to the desire to use materials from several sources at the same time, or at intervals. It appears wise, therefore, to include a provision in the specifications to make sure that changes will not be made during the course of the work, without the permission of the architect or engineer. The architect or engineer also wants to know that the materials used are of the quality specified, and facilities should be provided at the plant so that his representative can go there at any time and take the necessary samples for test. A control laboratory in which the usual material tests can be made, is an absolute necessity for the

proper operation of a ready-mixed concrete plant. This laboratory should be available for use by the representative or the engineer, so that he can promptly check up on the quality of the materials. It is common practice today in the steel industry to provide such facilities without charge.

The usual specifications for concrete, because of the necessity of allowing for many factors when concrete is made on the ordinary job, rightfully contain many restrictions on the quantities of materials and the water content. All that the engineer wants however, is concrete of a definite quality as determined by the strength and the workability. If the full advantage is to be realized from the development of the commercial concrete plant, the specifications must require the use of superior equipment and proper control at the commercial concrete plant, and then allow the plant to meet the requirements of quality without restriction as to the quantities used in the mix. The proposed tentative specifications were written with this idea in mind, and although such a specification may be considered premature, it is certainly a logical step forward.

The conditions surrounding the production of ready-mixed concrete are ideal for the use of proportions based on tests of the materials. The conditions surrounding the production of concrete on the ordinary job make it almost necessary to use established results for average materials. The commercial concrete organization must have available sufficient data from tests made by a reputable concrete testing engineer to enable the architect or engineer to determine that the quantities used will give the quality of concrete desired. Additional tests should be made by a competent testing engineer as a check on the quality of the concrete placed. This work should be done at the expense of the concrete producing organization who should receive copies of the test reports for their own use.

The only part of the commercial ready-mixed concrete operation which differs greatly from the ordinary job concreting procedure is that which has to do with the mixing and placing of the concrete. Ready-mixed concrete is essentially fabricated at such distances from the point of deposit as to require the use of vehicles other than barrows, buggies, or chutes for its transporta-

tion. The element of time is thus introduced into our considerations of the factors affecting the quality of the concrete from ready-mixed concrete plants.

The effect of time upon a number of the characteristics of concrete has been studied very thoroughly, but the effect of time on concrete as represented by delay, between the time of first adding water to the cement and the time the concrete is deposited in the forms, has not received much attention except in regard to the effect on the strength and workability. The work by H. F. Gonnerman and P. M. Woodworth is probably the most authoritative in regard to the effect of this delay on the strength.* Their conclusion is as follows: "The most striking result brought out by this investigation was the small loss in compressive strength due to standing for periods up to 6 hr. (protected from evaporation) of concrete remixed without the addition of water. When the mixtures remained plastic and workable the loss was practically nil. After the concrete ceased to be plastic the strength fell off rapidly. There was always a reduction in flow with an increase in standing period." These tests however were made under laboratory conditions using a cement which had an average initial setting time of 3 hr. 15 min. and a final set of 7 hr. and 27 min. Tests made on concrete as produced by commercial ready-mixed concrete organizations and reported in technical publications show results similar to those found by Gonnerman and Woodworth. A thorough consideration of the matter indicates, however, that in practice the time that can elapse between the addition of the water and the final deposition of the concrete, without deleterious effect on the strength, is influenced by the richness of the mix, the consistency, the characteristics of the aggregate, the agitation received either mechanically or by transportation, the characteristics of the cement, the surrounding temperature and humidity conditions, and the presence of admixtures. It is apparent that it is impossible at this time to write a specification giving limitations to all these factors. All that can be done is to require that the concrete be workable when received on the job and to set a conservative maximum limit on the time elapsing between the addition of the cement to the aggregates, and the time of placing.

*Tests of Retempered Concrete, Proc. Am. Concrete Institute, Vol. 25, p. 344.

The limit of one hour has been adopted in many cases, as engineers and architects consider some factor of safety necessary to cover our present lack of knowledge in regard to the effect of prehydration or very long mixing or the qualities of the concrete, other than strength. It is true however that such data as are available appear to be favorable to prehydration.

It is important to note, that the central proportioning-plant-truck-mixing systems, as well as the central mixing plant system, brings the cement in contact with water at the central plant, unless the materials are entirely dry, and therefore, any limitation of time should apply to both. Fortunately, it is generally not economically possible to transport the concrete for a distance that will require more than an hour haul.

The transporting of the concrete for appreciable distances tends to produce segregation of the materials. The wetter the mix, the more the tendency to segregate. Mixes of the consistency used for reinforced concrete work will segregate badly in a haul of one-half hour, which is an average for most commercial concrete plants. Concrete of the consistency used in road construction and heavy foundations can be transported in ordinary truck bodies for periods of over one hour without segregation. Agitator trucks are desirable for concrete of all types, as they help to prevent segregation, but as made at present, they do not efficiently handle concrete of one or two inch slump. Such trucks do not give sufficient mixing action to be considered as contributing to the mixing of the concrete, and therefore, the concrete from central mixing plants should be just as thoroughly mixed before loading on the truck as though there was no agitation.

Experience with central plant proportioned-truck-mixed concrete has shown that there is a tendency in certain types of mixers toward non-uniform distribution of the materials throughout the mixer. This is due to either improper charging, poor mixing action, or insufficient mixing, and can be corrected by proper care. The uusal truck mixer carries a load of from 3 to 5 cubic yards of concrete, and therefore, a mix of five minutes is not too long.

The specification, in all cases, refers to the time at which the cement was added, as a basis for determining the length of time

the hydration of the cement has been taking place. This was done so as to give a more definite basis, than to say "after the concrete was mixed" or "after the truck was loaded."

Proper servicing of the job is obviously necessary, and this means proper plant design so that no serious delay can occur due either to plant or material supply failure. The ideal condition would be for each organization to have several plants, or to have close cooperation between competing plants.

Architects and engineers are looking forward to the time when they can specify a given quality of concrete, and be as sure of getting it, with only nominal inspection, as they now are when they specify a given grade of steel. It may not be long before we have commercial concrete organizations furnishing concrete "erected" just as steel organizations supply and erect steel today. It is hoped that this report and the subsequent work of the committee will help to promote such progress in the field of concrete construction.

Readers are referred to the JOURNAL *for June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Specifications for the Small Job*

BY S. C. HOLLISTER†

The mischance that prevented the publication of this paper by Mr. Hollister in Volume 25 of Proceedings last June, presents the author's ideas on "owner insurance" of the quality of the concrete on the job that lacks constant supervision, at a time when a new Institute committee, 506, Concrete Specifications for the Small Job, is putting the finishing touches on a similar document: "Construction Specification for Concrete Work on the Small Job." Mr. Lord as author-chairman made the original draft and the report has gone to critic members of the committee for further consideration. It is scheduled for early publication. The committee will now have the benefit of Mr. Hollister's paper.*—EDITOR

INTRODUCTORY

THE PURPOSE of this paper is to propose for discussion a typical specification easily adaptable to the small job. Such a job differs from others in several important respects. It does not attract the larger contracting organization with its staff of trained men. There cannot be a resident engineer, although there should be a resident inspector paid by the owner and under the engineer's direction. There is seldom an opportunity to exercise the control over the manufacturer of concrete through field specimens followed by subsequent adjustments, which are becoming common on large jobs. And yet, there is desired by engineer and owner the same high grade of construction expected and obtained on the large job.

Specifications must perform several functions. In the first place, they must be descriptive of the extent and quality of the

*This paper, on the program of the 25th annual convention, was, in the author's absence, presented by title only. The author's illness prevented revisions he desired to make in time for publication in Vol. 25.

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work contemplated. They should be the clear outline necessary for mutual understanding and equitable performance of the work on the part of both engineer and contractor. They must often explain particular processes and procedures. The bidder must have full understanding of what is expected of him and at the same time the reasonable authority of the engineer in discharging his responsibilities must be made clear.

The specification proposed here is based on that of the Joint Committee but shortened in accordance with the needs and restrictions of the small job. It is neither new nor original but rather is based on all the data the writer has been able to bring to bear on the subject.

Each class of constructive work, and each community as well, has its own group of available contractors. It is manifestly impossible to draft a single specification which will meet all these variable circumstances. It is hoped, however, that the form here suggested will serve as a guide to the engineer in preparing a specification to meet his immediate needs, and in many instances will provide a usable form as it stands. It must, of course, be preceded by the legal clauses involving the extent and administration of the contractual relation.

CONSTRUCTION SPECIFICATIONS

GENERAL

(1) The work hereunder shall include furnishing all labor and materials in the construction of

(Here should be inserted a brief description of all the construction to be done under these specifications.)

SCOPE OF WORK

(2)

(Here should be outlined where the work begins and where it ends; interrelation with property lines, streets, existing structures; replacements, where necessary to remove existing work for facility in installing new work; and all other general items which fix the confines and inter-locking features of the work.)

PROVISION FOR UTILITY LINES

(3)

(Care of utility lines in the line of the contemplated work should be set forth here. Supply of power, water, etc., for the new work should be outlined, and at whose expense it shall be installed.)

PLANS

(4) The plans which illustrate the work hereunder and which

are included in these specifications and made a part hereof are as follows:

(List all contract drawings by number, title and date. Also provide drawings to be supplied at a future date amplifying the work shown on the above drawings.)

PRELIMINARY PREPARATIONS

(5)

(Describe in detail all preliminary preparations, obtaining of permits, clearing site, etc., pertinent to the work hereunder.)

LOCATION OF CONTRACTORS PLANT

(6) The Contractor shall, before proceeding with the work, obtain the approval of the Engineer, in writing, for the placing of his plant as regards the ground to be occupied during the period of the work hereunder. Private property not controlled or granted by this owner shall be occupied at the sole expense and responsibility of the Contractor.

INSPECTION AND SAMPLING OF MATERIALS AND WORKMANSHIP

(7) All materials to be incorporated in the construction hereunder shall be subject to inspection and sampling after delivery to the site and the Contractor shall furnish such samples and shall provide facilities for adequate inspection at his own expense throughout the progress of the work.

(8) The Contractor shall furnish sufficient space in his field office for the Inspector and shall furnish him with free telephone service.

STORAGE AND INSPECTION OF MATERIALS

(9) All materials to be incorporated in the work hereunder shall be so stored and furnished with such protection as to prevent their deterioration or impairing of their quality.

CEMENT

(10) All cement used hereunder shall be portland cement and shall conform to "Standard Specifications and Tests for Portland Cement" (serial designation C-9-26*) of the American Society for Testing Materials.

FINE AGGREGATE

(11) Fine aggregate shall consist of sand having clean, hard, strong, durable, uncoated grains, free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter,

*Or subsequent serial designation.

lumpy, or other deleterious substances.

(12) Fine aggregate shall range in size from fine to coarse within the following limits:

Passing through No. 4 sieve not less than eighty-five (85) per cent by weight.

Passing through No. 50 sieve not more than thirty (30) per cent nor less than ten (10) per cent, both by weight.

(Clean sand, free from silt or clay or dirt of any kind is of prime importance. For testing sand for organic impurities see "Abrams-Harder Field Test," A. S. T. M., XIX, Pt. I, p. 321 (1919); and for gradation, A. S. T. M., Std. C41-24. Effect of admixtures on strength of concrete is discussed in Bull. 8, P. C. A., 1925. In general, inert admixtures which swell when wet and shrink in drying, such as clay, may have undesirable effect on the durability of concrete exposed to weather, while admixtures not so affected, in proper amounts, may aid in placing and rendering dense concrete. Sand from sources of proven reliability is always preferable, in the absence of thorough test results.)

COARSE AGGREGATE

(13) Coarse aggregates shall consist of crushed stone, gravel, or blast furnace slag, having clean, hard, strong, durable, uncoated particles, free from injurious amounts of soft, friable, thin, elongated, or laminated pieces, alkali, organic or other deleterious matter.

(14) Coarse aggregate shall range in size from fine to coarse within the following limits, except for cyclopean concrete or for making balustrade spindles:

Passing a one (1) inch sieve not less than ninety-five (95) per cent by weight.

Passing a No. 4 sieve not more than ten (10) per cent by weight.

Passing a No. 8 sieve not more than five (5) per cent by weight.

(The stone must be *clean*, free from coatings of silt, clay, etc. Stone that deteriorates when exposed to the elements should not be used. It may be tested for organic impurities by the Abrams-Harder test. The grading given here will suffice for general building work. Special grading will be required for special work, as determined by reinforcement clearances, extent of concrete mass, etc.)

CYCLOPEAN AGGREGATE

(15) Cyclopean aggregate for use in cyclopean concrete, in addition to the size and gradation specified above, may range in size from three (3) inches in diameter to pieces weighing one hundred (100) pounds each and shall consist of clean, hard, durable stone or gravel and shall be so placed that each stone is entirely surrounded by the concrete in which this aggregate is being embedded. It shall be used only in those members indicated on the plans.

STORAGE OF AGGREGATES

(16) Aggregates shall be stored as to avoid inclusion of foreign materials. Frozen aggregate or aggregate containing lumps of frozen material shall be thawed before using.

WATER

(17) Water for concrete shall be clean and free from injurious amounts of oil, acid, alkali, organic matter or other deleterious substances.

STEEL REINFORCEMENT

(18) Steel reinforcement shall meet the requirements of the "Standard Specification for * Concrete Reinforcement Bars" of the American Society for Testing Materials for * grade.

(19) Deformed bars shall be of a form which shall be approved in writing by the Engineer.

EXCAVATION OF FOUNDATIONS

(20) Excavation shall be carried down to the depth indicated on the plans and shall be of such extent as is required by the plans to receive the footings. Adequate shoring shall be provided as may be necessary to prevent damage to adjoining property or injury to workmen. Materials removed in excess of that needed for backfill above the completed footings, shall be removed from the site by and at the expense of the Contractor.

PREPARATION OF FOUNDATION BED

(21)
(Full description of preparation of the contact bed should be given.)

FORMS

(22) Forms shall conform to the shape, lines and dimensions of the concrete as called for on the plans. Lumber used in forms for exposed surfaces shall be dressed to a uniform thickness, and shall be free from loose knots or other defects. Joints in forms shall be horizontal or vertical. For unexposed surfaces and rough work, undressed lumber may be used. Lumber once used in forms shall have surfaces to be in contact with concrete thoroughly cleaned before being used again. All interior angles not occurring

*Insert here the grade of reinforcing steel desired. Usually intermediate grade steel is carried in stock by bar companies.

on an exposed face of the structure shall be formed with a one (1) inch bevelled fillet, unless noted on the plans.

(23) Forms shall be substantial and sufficiently tight to prevent leakage of mortar;* they shall be properly braced or tied together so as to maintain position and shape. If adequate foundation for shores cannot be secured, trussed supports shall be provided.

FORM TIES

(24) Bolts and rods shall preferably be used for internal ties; they shall be so arranged that when the forms are removed no metal shall be within one (1) inch of any surface. Wire ties will be permitted only on light and unimportant work; they shall not be used through surfaces where discoloration would be objectionable. Shores supporting successive stories shall be placed directly over those below, or so designed that the load will be transmitted directly to them. Forms shall be set to line and grade and so constructed and fastened as to produce true lines. Special care shall be used to prevent bulging. Clearances shown on the plans for metal reinforcement shall apply also to metal ties left in the concrete.

FORM TREATMENT

(25) The inside of forms shall be coated with non-staining mineral oil or other approved material. Where oil is used it shall be applied before the reinforcement is placed.

CLEAN-OUTS

(26) Temporary openings shall be provided at the base of column and wall forms, and at other points where necessary to facilitate cleaning and inspection immediately before depositing concrete.

CLEANING OF STEEL REINFORCEMENT

(27) Metal reinforcement, before being positioned, shall be thoroughly cleaned of mill and rust scale and of coatings that will destroy or reduce the bond. Reinforcement appreciably reduced in section shall be rejected. Where there is delay in depositing concrete, reinforcement shall be re-inspected and, when necessary, cleaned.

*The writer realizes the advantages in "bleeding" the concrete while still in the plastic state through forms designed for the purpose, in order to obtain a denser, stronger concrete; but such methods are not sufficiently understood by builders to justify their use on the small job.

BENDING

(28) Reinforcement shall be carefully formed to the dimensions indicated on the plans. Cold Bends shall be made around a pin having a diameter of eight (8) or more times the least dimension of the reinforcement bars.

(29) Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement will be permitted only when the entire operation is approved by the Engineer.

PLACING

(30) Steel reinforcement shall be accurately positioned and secured against displacement by using annealed iron wire of not less than No. 18 gauge, or suitable clips at intersection, and shall be supported by concrete or metal chairs or spacers or metal hangers. Clearance between forms and reinforcement indicated on the plans shall be rigidly adhered to and no corrodable metal, whether for reinforcement or otherwise, shall be nearer the forms than this specified clearance distance.

SPLICING

(31) Splicing of reinforcement shall be accomplished by lapping bars not less than forty diameters. The lapped portions shall be spaced a clear distance apart not less than one and one fourth ($1\frac{1}{4}$) times the maximum size of the coarse aggregate to be used in the concrete at that point. Splicing of bars arranged in parallel system shall be made at staggered locations and in such manner that not more than twenty (20) per cent of the bars in the parallel system shall have splices on any line across the system.

PROPORTIONING

(32) The unit of measure shall be the cubic foot. Ninety-four pounds of cement (one bag of $\frac{1}{4}$ bbl.) shall be considered as one cubic foot.

(33) The method of measurement shall be such as to secure the required proportions in each batch. The aggregate shall be measured by volume. In volume measurement the fine aggregate and the coarse aggregate shall be measured loose, as thrown into the measuring device and struck off. The water shall be so measured as to insure the desired quantity in successive batches.

(34) Concrete shall be mixed in the proportions indicated in the table in Sec. 35, except as modified by the Engineer under Sec. 37.

(35) The design of the various portions of the structure is based on the assumption that the concrete will develop the compressive strength at twenty-eight (28) days which, for the purpose of design only, are given in the following table:

PROPORTIONS OF CONCRETE

Class	Part of Structure	Proportions by Volume			Concrete Strength Assumed as Basis for Design, lb. per sq. in. at 28 days
		Cement D	Fine Aggregate	Coarse Aggregate	

(This table should set forth the classes of concrete to be used in the structure, with typical proportions established by the Engineer through previous tests on the kind of material to be used. In the absence of such tests see Spec. for Conc. and Rfcd. Conc., Joint Committee, App. 16, proc. A. C. I., (1925) p. 404.)

(36) Variations in the grading of the aggregates from that given in Secs. 12 and 14, and on which the above proportions were based, may be made upon the approval of the Engineer and in such proportions as he may direct, but no claim shall be made for extra compensation therefor.

(37) The Engineer shall have the right to make any changes in proportions of materials that may be necessary or desirable, and adjustment of compensation will be made in accordance with the provisions of the schedule of "Extra Work or Deduction of Work"* of this contract. Frequent tests will be made by the Engineer at the expense of the owner throughout the work to determine whether the concrete produced by the proportions given in the foregoing table is of the quality assumed in the design. The Contractor shall provide such facilities as may be practicable for carrying out tests and shall cooperate in every way to the end that concrete of the desired quality shall be obtained.

CONSISTENCY

(38) The quantity of water used shall be the minimum necessary to produce concrete of a workability required by the Engi-

*As a part of the Proposal Form there should be provision for unit-price bids on furnishing (or deducting) various materials employed in the work as a means of fixing compensation or deductions as required during the progress of the work.

neer. The consistency of the concrete shall be measured by the slump test as described in the "Tentative Method of Test for Consistency of Portland Concrete for Pavements or for Pavement Base" (serial designation D138-2-T) of the American Society for Testing Materials. The slump for the different types of concrete shall not be greater than that indicated in the following table unless authorized by the Engineer.

WORKABILITY OF CONCRETE

Class of Concrete	Maximum Slump In.

(The slumps required by the Engineer should be listed here, in accordance with the classes of concrete and placing requirements.)

(39) The consistency shall be checked from time to time during the progress of the work.

MIXING

(40) The mixing of concrete, unless otherwise authorized by the Engineer, shall be done in a batch mixer of approved type which will insure a uniform distribution of the materials throughout the mass, so that the mixture is uniform in color and homogeneous. The mixer shall be equipped with suitable charging hopper, water storage, and a water-measuring device. The entire contents of the drum shall be discharged before recharging. The mixer shall be cleaned at frequent intervals while in use. The volume of the mixed material per batch shall not exceed the manufacturer's rated capacity of the mixer.

(41) The mixing of each batch shall continue not less than one minute after all the materials are in the mixer, during which time the mixer shall rotate at a peripheral speed of about two hundred (200) feet per minute.

HAND MIXING

(42) When hand mixing is authorized by the Engineer it shall be done on a water-tight platform. The cement and fine aggregate shall first be mixed dry until the whole is of a uniform color. The water and coarse aggregate shall then be added and the entire mass turned at least three times, or until a homogeneous mixture of the required consistency is obtained.

RETEMPERING

(43) The retempering of concrete or mortar which has par-

tially hardened, that is, remixing with or without additional cement, aggregate or water, will not be permitted.

CLEANING OF MIXER AND CONVEYING EQUIPMENT

(44) Before beginning a run of concrete, hardened concrete and foreign materials shall be removed from the inner surface of the mixing and conveying equipment.

CLEANING FORMS

(45) Before depositing concrete, debris shall be removed from the space to be occupied by the concrete; forms shall be thoroughly oiled. Reinforcement shall be thoroughly secured in position and approved by the Engineer. (See Secs. 25 and 30.)

CONVEYING CONCRETE

(46) Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which shall prevent the separation or loss of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. It shall be so deposited as to maintain, until the completion of the unit, a plastic surface approximately horizontal. Forms for walls or other thin section of considerable height, shall be provided with openings, or other devices, that will permit the concrete to be placed in a manner that will avoid accumulations of hardened concrete on the forms or metal reinforcement. Under no circumstances shall concrete that has partially hardened be deposited in the work.

(47) When concrete is conveyed by chuting, the plant shall be of such size and design as to insure a practically continuous flow in the chute. The angle of the chute with the horizontal shall be such as to allow the concrete to flow without separation of the ingredients. The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run; the water used for this purpose shall be discharged outside the forms.

PLACING CONCRETE

(48) Concrete, during and immediately after depositing, shall be thoroughly compacted by means of suitable tools. For thin walls or inaccessible portions of the forms, where rodding or

forking is impracticable, the concrete shall be assisted into place by tapping or hammering the forms opposite the freshly deposited concrete. The concrete shall be thoroughly worked around the reinforcement and around embedded fixtures, and into the corners of the forms.

UNWATERING

(49) Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the Engineer. Any flow of water into the excavation shall be diverted through proper side drains to a sump, or be removed by other approved methods which will avoid washing the freshly deposited concrete. Water vent pipes and drains shall be filled by grouting or otherwise, after the concrete has thoroughly hardened.

SURFACE PROTECTION

(50) Exposed surfaces of concrete shall be protected from premature drying for a period of at least seven days after being deposited.

CONCRETING IN FREEZING WEATHER

(51) Concrete when deposited shall have a temperature of not less than 40° F. nor more than 120° F. In freezing weather suitable means shall be provided for maintaining the concrete at a temperature of at least 50° F. for not less than seventy-two (72) hours after placing, or until the concrete has thoroughly hardened. The methods of heating the materials and protecting the concrete shall be approved by the Engineer. Salt, chemicals or other foreign materials shall not be mixed with the concrete for the purpose of preventing freezing unless approved by the Engineer.

CONTINUOUS PLACING

(52) Concrete shall be deposited continuously and as rapidly as practicable until the unit of operation, approved by the Engineer is completed. Construction joints at points not provided for in the plans shall be made in accordance with the provisions in Section 53 and 58.

PLACING AGAINST HARDENED CONCRETE

(53) Before depositing new concrete on or against concrete which has set, the forms shall be retightened, the surface of the

set concrete shall be roughened as required by the Engineer, thoroughly cleaned of foreign matter and laitance, and saturated with water. The new concrete placed in contact with hardened or partially hardened concrete, shall contain an excess of mortar to insure bond, and shall not be placed until the saturated surface has become dull through absorption or removal of all excess water.

CYCLOPEAN CONCRETE

(54) Cyclopean aggregate shall be thoroughly embedded in the concrete. The individual stones shall not be closer to any surface or adjacent stone than the maximum size of the coarse aggregate in which the cyclopean aggregate is embedded. Stratified stone shall be laid on its natural bed.

REMOVAL OF FORMS

(55) Removal of forms shall be undertaken only after the member cast therein shall have acquired sufficient strength to support safely its own weight and the weight of members it, at that time, supports.

DEFECTIVE CONCRETE

(56) Any defective work discovered after the forms have been removed, which in the opinion of the Engineer cannot be re-repaired satisfactorily, shall be removed at once and replaced. No compensation will be allowed for patches or replacements because of defects. Defects shall include honeycombing, or other surface blemishes; or portions which through lack of uniformity in the concrete render them, in the opinion of the Engineer, inadequate in strength. Improper alignment of outside faces shall be considered defective work.

PATCHING

(57) Patching where permitted shall be done only under the supervision of the Engineer. The cavity to receive the patch shall be wetted thoroughly and allowed to stand sufficient time for the surface of the cavity to become dull but not lightened in color. The patching material shall be of the same material and in the same proportions as the concrete receiving the patch. It shall be worked into place in a dry consistency and shall be thoroughly rammed. The patch shall be built up and supported in such a manner as will prevent sagging.

CONSTRUCTION JOINTS

(58) Joints not indicated on the plans shall be so designed and located as least to impair the strength and appearance of the structure. To prevent laitance in horizontal joints, excess water shall be removed from the surface forming the joint after depositing the concrete. Surfaces of contact shall be cleaned and wetted before depositing is resumed, and any laitance shall be removed. Where additional resistance to horizontal shear is required, stones shall be partly embedded in such a manner as to key with the adjoining concrete; or mortises or keys shall be formed in the concrete.

SURFACE FINISH OF SHOWING FACES OF BRIDGE

(59) The whole of a showing face between prescribed construction joints shall be cast in one continuous operation. Construction joints, when not shown on the plans, shall be made as directed by the Engineer, and shall be true to line with sharp unbroken edges.

(60) The same brand of cement, and the same kind and size of aggregate, shall be used throughout the whole of any showing face. The aggregate shall be approved by the Engineer as to character and color.

(61) For showing faces the forms shall be smooth and watertight. If wood be used, the boards shall be planed, grooved, and tongued, evenly matched and tightly placed. They shall be so constructed as to be removable in sections by unscrewing or otherwise loosening them without hammering or prying against the face. Any offsets in the forms that may occur shall be smoothly dressed and any openings pointed flush with stiff clay or plaster of Paris in order to prevent leakage or the formation of fins.

(62) Great care shall be exercised to obtain a uniform distribution of coarse aggregate next to a showing face. The concrete shall be worked during placing to force the coarse aggregate toward the form of the showing face.

(63) Face forms shall be removed as soon as practicable in order to facilitate effective repair of void spaces or broken corners before the surface has dried. Care shall be taken to avoid roughening or injuring corners, and to keep edges sharp.

(64) As soon as the face forms are removed any fins or other

projections shall be carefully removed, and offsets leveled, and any voids or damaged places shall immediately be saturated with water and filled with a mixture of the same composition as that used in the surface, and brought even with the surface by means of a wooden spatula or float. A steel trowel shall not be used to finish the surface. The face shall be finished free from streaks, discolorations, or other imperfections. Plastering will not be permitted.

SCRUBBED FINISH

(65) Where so designated on the plans, all exposed or showing surfaces shall be given a scrubbed finish. The forms must be removed and the scrubbing done before the concrete has hardened. The surface shall be scrubbed with fiber or wire brushes using water freely, until the surface film of mortar is removed and the aggregate uniformly exposed; then rinsed with clean water. If portions of the surface have become too hard to scrub in equal relief, dilute Hydrochloric acid (one (1) part acid to four (4) parts water) may be used to facilitate the scrubbing. The remaining acid shall be thoroughly removed with clean water.

BLOCK FINISH

(66) All surfaces indicated on the plans as having block finish shall be rubbed to a smooth even surface and color by rubbing with a carborundum block and clean water without the addition of cement or other material.

Readers are referred to the JOURNAL *for June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Recommended Practice for the Manufacture of Concrete Block and Building Tile

A Preliminary Review

BY P. M. WOODWORTH, AUTHOR-CHAIRMAN COMMITTEE 708

Committee 708 was asked to review critically the Tentative Recommended Practice for the Manufacture of Concrete Building Block, Building Tile and Brick in the light of the rapid developments in the industry in the last few years and present to the 26th annual Convention a preliminary report which might serve to develop the discussion and the further data needed for a complete revision of the 5-year old document. Here will be found emphasis on serious new factors which had not become conspicuous in the industry half a decade ago. This is on the program for Convention discussion.—EDITOR

SINCE former Committee P-6 presented proposed Recommended Practice for the manufacture of Concrete Building Block, Building Tile and Brick adopted as tentative in 1925 (P-6—A-25T), the production of concrete block has more than doubled. One of the principal factors in this marked growth has been the increasing use of concrete masonry for above grade construction. With this development a number of physical properties of the concrete unit, in addition to strength, absorption, and permanence, are of increasing importance. These factors are fire resistance, sound insulation, heat insulation at low temperatures, shrinkage of wall sections, nailability surface texture and weight per square foot of wall area. New methods of molding, proportioning, mixing

and curing have been proposed and used to develop in one way or another certain of these important properties or to effect manufacturing economies. The use of lightweight aggregates of a porous nature, either natural or manufactured, to furnish these desirable properties also has necessitated the development of a new technique in certain of the manufacturing methods.

The following comments on the 1925 report of Committee P-6 are made solely for the purpose of stimulating discussion in order that the recommendation may be revised and made representative of the best practice followed today.

The Committee believes that a recommended practice for the manufacture of concrete brick should be developed separate from that covering concrete block and tile. This recommendation is made for the following reasons:

1. Concrete brick specifications are only tentative and considerable difficulty is being experienced in making further progress.
2. Relative interest in concrete brick is decreasing.
3. Research work would be greatly increased if brick were included thus retarding progress in concrete block and tile.
4. Desirable properties in concrete brick are frequently different from those of block and tile.

Various divisions of the 1925 report are herewith discussed in view of changes in plant practice and more recently developed information.

MATERIALS

B—Aggregates—In the 1925 report no mention is made of burned shale or cinders as suitable aggregates for concrete units other than in the general clause "other approved inert materials." The widespread and growing use of these aggregates for the manufacture of lightweight units indicates that they deserve more than passing mention.

It is questionable whether the phrase "strong, durable grains" is descriptive of an aggregate such as cinders or Haydite. In addition a limit should be set up for the combustible content of the cinders. On the basis of the tests at the Underwriters' Laboratories, 20 per cent has been fixed as the limit to receive the Inspection Service for a three-hour fire retardant rating. It is

possible that this per cent should be increased. The use of the colorimetric test as a basis for rejection of a fine aggregate has never been practiced and might well be deleted from the main part of the report. As described in the notes, the colorimetric test is only an indication that further tests on the aggregate should be made.

E—Proportions—Proportions had received considerable study. Manufacturers are finding that the building trade in different sections is showing a marked preference for units with definite surface characteristics. These preferences vary from the rough coarse texture, used in back-up and in interior walls, to the smooth fine sand finish for exposed work. Coarser or finer gradings, as the case may be, have been chosen to furnish this texture. In general the coarsest grading in line with the desired surface texture and density will yield the most economical units where fine and coarse aggregates are equal in cost.

The general recommendations concerning the proportioning of aggregate in this report are still standard practice. The increased use of concrete products for above grade construction has created a demand for products with a comparatively rough texture. This demand is an outgrowth of the practice of applying either stucco or plaster directly to the unit. In the manufacture of light-weight units, the use of smaller amounts of fines, i. e., material passing a fifty mesh sieve is becoming general in some localities. This is being done with the thought that lighter weight units are obtainable in addition to rough texture. Uniformity of grading is an increasingly important feature in the production of block. Proportions must be controlled so that a unit is produced which will meet the American Concrete Institute or local building code requirements economically with the desired surface texture.

The strength specifications for heavy and non-load bearing units have been dropped as such, but the same intent is still written into the specification for concrete block and tile. The manner of stating the absorption requirement should be simplified so that reference to unit weight of the product is unnecessary. The simple method of stating absorption requirements in pounds per unit might well be incorporated in the block and tile specifications.

In this section of the code, the absorption requirements could be more simply stated as follows:

Building block or tile shall not absorb more than 14 pounds of water per cubic foot of oven dry concrete.

For an average three-core 8 x 8 x 16 concrete unit with 36% air space this absorption is equivalent to 5 pounds per unit irrespective of the type of aggregate or the weight of block.

This would delete the qualifying clause regarding absorption limits and unit weight and serve to clarify this part of the specification.

MANUFACTURE

A—Consistency—The most desirable consistency for tamped products—"as wet as possible"—has had almost universal adoption in products plants for the commonly used aggregates. In the manufacture of lightweight units, however, the tendency is to make the block a trifle drier in order to obtain the desired rough surface texture which will have excellent bonding qualities. Also where very fine material is used, that is, aggregate with a fineness modulus of less than 2.80, too much water can be included to get maximum strength. There has been a considerable development during this period in the utilization of high frequency vibration to consolidate the concrete. Drier consistencies are used in conjunction with slightly longer vibration periods. Further studies should be made to determine the most desirable consistency to be used with this method and to learn how to take full advantage of this method of compacting concrete.

B—Mixing—In general the mixing time has been gradually increased during the past five years. Plants using a total mixing time of about five minutes are greatly in the majority. Indications are that some plants are going too far in the length of time of mix; not that the concrete is injured but that the additional cost of mixing is too great. In the manufacture of lightweight units it is claimed that the technique of mixing has important influence on the results obtained. Split time mixing, that is, mixing the aggregates and water either all, or in part, for a one or two minute period before the addition of cement has found many advocates. The advantage of this practice is explained on the basis that a lighter and stronger block is being obtained by filling the vesicular cavities with water instead of cement paste. Mixing recom-

mendations for the different types of aggregate should be formulated.

C—Compacting—There has been no material change in the principle for equipment for making a block by the tamp process. Machinery has been developed which has speeded up production, however. The conclusions reached in the investigation by Committee P-6 on the effect of tamping should be studied and incorporated in this report. There have been a number of vibration machines placed on the market in the last five years. A vibration speed of about 3,000 to 3,500 R. P. M. is general. Some advocate higher speeds. The period of vibration is varied between ten to fifty seconds to suit the consistency of the mix, the type of units and type of machine. The importance of this method of compacting should not be underestimated as it offers a number of advantages where strong dense concrete is desired.

D—Curing—With developments in the use of concrete masonry units in above grade construction, for back-up, enclosure and partition walls, the method of curing the units has assumed increased importance. The use of wet or green block in this type of construction will tend to produce shrinkage cracks which are objectionable. In order to develop a desirable strength, a period of moist or steam curing is necessary. This should be followed by a definite drying out period in order that the units may be delivered to the job in an air dry condition.

The relative value of different methods of curing has long been a live question. Numerous investigations, mostly with low pressure steam and fog spray systems have developed standard practices for these two methods. Recent investigations indicate that steam curing at high temperatures and low pressures are detrimental to the ultimate strength of the concrete. On the other hand, high pressure steam curing similar to the treatment given sand lime brick has been found to have a decidedly beneficial effect, especially upon early strength. The use of this method, however, has not been general in products plants, first because the initial cost for equipment is comparatively high; second because of the meager information as to the value of this method of curing.

Fuel costs for high pressure steam curing have been found to be approximately that of low pressure steam curing. High pressure

steam curing may offer considerable possibilities as a curing method in the development of the ultimate strength of the product at an early age, thus reducing the necessity for large stock piles.

Another method of curing consists of the use of air at a temperature close to 212° F. with as high a humidity as it is possible to maintain. The heating unit utilizes either a good grade of coke or an oil burner, and is directly connected to the curing chamber. All the warm air and combustion gases are drawn into the curing room and then re-circulated through the heating system by means of a large fan. Small atomizers are connected to a hot water coil in the furnace and spray moisture into the curing room. It is said that by this method, high strength and dry units can be obtained after eight to twelve hours of curing without any detrimental effects.

Shrinkage—The extent of shrinkage in any masonry wall after erection is an important factor. Excessive shrinkage may cause the formation of unsightly cracks. For the most part, cracking in concrete masonry walls is due to faulty construction such as inadequte footings or improper bonding between different types of masonry and structural members. There are instances, however, where cracks were probably caused by shrinkage in the wall. Preliminary studies have indicated that a large portion of the shrinkage, occurring in a concrete masonry wall, can be eliminated by the use of air dry units. The methods used by the products manufacturer in curing and subsequent treatment prior to erection have an important bearing on the shrinkage. Such high pressure steam curing studies on concrete products as have been made indicate that shrinkage is largely eliminated when products are cured by this method. Different curing methods and treatments should be studied, for their effect on strength, shrinkage and reduction of cost of manufacture.

Temperature Insulation—Reliable information regarding the insulating properties of concrete block, especially those made with sand-gravel or limestone, is not available. The assumptions are usually based on 90% of the heat loss of a plain concrete wall.

Cinders and Haydite, because of their cellular structure, impart a considerably higher insulating value when used in the concrete unit. It has been found that the density of the material is a

major factor in determining the insulation properties of a material. Tests should be conducted to develop data regarding the insulating qualities of different gradings of all the commonly used aggregates. These data should also include studies on the effect of different types of walls of which an 8-inch concrete unit is the base. Some studies made to date indicate that those gradings of aggregates which may give the best results for one condition, for example fire resistance, may not be best for other results such as sound insulation, or even heat insulation at lower temperatures.

Sound Insulation—The sound insulation characteristics of a concrete masonry wall are another factor where sufficient data of an authoritative nature are not available. The effect of grading and surface characteristics of the units on the sound absorption properties should be determined.

SUMMARY

As outlined in the above discussion, further research into manufacturing details is necessary so that the processes, whereby desirable qualities of a concrete unit are enhanced, will be standardized. The principal factors to be studied are as follows:

1. Manufacturing technique for different types of aggregate.
2. The effect of different curing methods upon early strength, shrinkage and cost.
3. The effect of grading on sound insulation and acoustics.
4. Evaluation of all desirable properties of concrete masonry to produce a unit that gives the most satisfactory results for the different uses to which it is to be put.
5. Factors affecting weather resistance and the development of an accelerated weather resistance test.

Readers are referred to the JOURNAL *for June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Design of Concrete Products Plants for Single or Multiple Shift Operation

BY BENJ. WILK*, AUTHOR-CHAIRMAN COMMITTEE 707

The subject assigned to Committee 707 is not one that lends itself to the usual form of report. Plant operation on double shift is with a few exceptions a theory in the products field and a majority committee report is not the end sought. But for these exceptions "double shift" is a potential development. Mr. Wilk has here furnished the foundation of a discussion—a discussion which necessitates some analysis of operation and it should serve as the basis of contributions that will bring up operating conditions for closer scrutiny in the convention where this report will have formal presentation and further in the pages of this JOURNAL. *The report has been referred to several competent critics to start the discussion. Before multiple shift plants become general—if they ever do—the entire operating program may be improved by analysis.*—EDITOR

INCREASED use of concrete products during the past ten years has led to the establishment of a number of large plants each producing over a million units annually. These new plants have usually been well equipped. At the same time many smaller plants have increased their production without adding greatly to their plant equipment.

These two developments have helped to bring to the front a question that seriously effects the design of concrete products plants for economical production. That question is: Shall a plant be designed for single or multiple shift operation?

From a theoretical standpoint there is much to be said in favor of multiple shift operation. A plant that is equipped to produce

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500,000 units annually, with a single shift, from a theoretical point of view could produce one million units with two shifts. This would mean cutting the overhead costs practically in two, which in many cases would amount to a large item, and frequently would mean the difference between success and failure.

Looking at this problem, however, from a practical standpoint, the picture is quite a bit different.

In order to bring the problem to a working basis we will assume we are going to place a plant in a market where we want to produce one million equivalent eight inch block annually, and the plant is to operate 250 days per year to secure this volume. This market would require that we make twelve inch block, eight inch block, rockface block, specials and slabs.

DESIGN OF PLANT

One million units per year of 250 days, or 4,000 per day, each weighing 50 lbs. means an average of at least 40 lbs. of aggregate per unit, or 160,000 lbs. of aggregate per day. Allowing for fifty per cent for peak demand means 240,000 lbs., or 120 tons per day, the equivalent of slightly more than two carloads of 50 tons each.

Shall we have a siding, conveyors, and overhead bins, or shall we handle solely with trucks.

In our proposed market we find the spread between truck delivery and rail delivery to be approximately twenty-five cents per ton. Rail delivery is available only seven months a year. Using 80 tons average per day, the twenty-five cent per ton saving means twenty dollars per day, over the rail delivery period of 175, or $3,500.00 per year. Figuring twenty per cent for depreciation, interest on investment and repairs, anything under $17,500.00 for bins, conveyors and siding, would be spent to advantage. This does not take into account the saving in handling aggregates from over-head bins as compared with other methods of handling.

SELECTION OF MACHINES

To obtain one million units annually, we shall assume for simplicity's sake, two stripper machines on twelve inch block giving an output of 1,600 twelve inch or 2,400 eight inch equivalent per day; one eight inch stripper machine giving 1,000

eight inch block per day; and one downface machine for rockface block, slabs and specials, producing 600 units per day, a total of 4,000 equivalent units per day.

This choice of equipment is based on actual experience where we have found that an economical method of producing block is to have one man on a machine, making, off-bearing, and placing in curing rooms. This may sound uneconomical, because the machine is not working to its fullest daily capacity, but observation in other plants, as well as in our own, proves that the one man per machine idea, for day in and day out operation in a well laid out plant, is a low cost method of production. It is not necessarily the only low cost method of production. In fact in plants manufacturing more than one million equivalent units annually, larger machines have proved satisfactory.

To supply the four machines assumed for our proposed plant, we will use overhead bins and two mixers so placed that we can dump into two machines from each setup.

For curing 4,000 equivalent eight inch block daily, we would require at least six curing rooms each eight feet wide and sixty feet long.

SINGLE OR MULTIPLE SHIFT

This brings us directly to the single or multiple shift question. In determining where we can save in the design, let us start with the unloading. Any conveying equipment worthy of the name will unload two to three cars of aggregate in a single shift. Therefore, in one shift we can unload all of the raw materials we need daily for our plant.

Now as to the machines. The rockface, slabs and specials demand is not large and only one shift a day would be necessary to take care of this demand. An average of one thousand eight inch block, or only one shift will take care of the eight inch demand. The twelve inch production of 1,600 daily can be taken care of either with two machines running only one shift, or one machine running two shifts. This means that if we decide to use a night shift we will run only one twelve inch machine during the night, but we will save an investment of one twelve inch machine costing about twenty-five hundred dollars.

The number of pallets and racks required would be the same for both the single and double shift plants, but there would be the

disadvantage in the two shift plant of taking out pallets during the night if twenty-four hours curing were the general practice. If night piling were not done then more curing rooms, pallets and racks would be required. With up-to-date curing methods we will assume that all of the block in the two shift plant can be taken out during the day shift hours.

In analyzing night work we have found that to get good men to work at night we have had to pay a bonus of ten per cent above the day rate, even on a piece work basis. Repairs have been greater during the night shift, and there has been more breakage. Men are not as regular in attendance at night as they are during the day. One man missing from a small night crew disorganizes the entire crew, causing a decrease in production. Sometimes one absentee means that the remainder of the crew will not work that particular shift. Frequently the night shift, especially with green men, will so misuse a machine that it will take several hours in the morning to get it ready for the next shift.

If these troubles are to be avoided then it is necessary to have as efficient night supervision as day supervision. This costs money, and for one machine producing 800 twelve inch block per night, or less than one-third of the entire day's production, such supervision would be uneconomical.

Again, at the best, night shifts on the two shift plan would work only six to eight months per year, and it would be difficult to recruit a new and efficient gang for night work after men have been laid off from four to six months.

Apparently, the only definite saving in a multiple shift plant such as we are studying, is in the single machine costing $2,500.00. Eight per cent on this investment is $200.00. This is a very small saving as compared to the extra cost of working a night shift.

A very recent investigation among large automobile plants in Detroit, where night work is a common practice and frequently continues almost all year, has brought the following comments:

"Day shift has about twenty per cent less scrap, and accidents are much lower on day shifts than on night shifts."

"Absentees run higher on the night shifts."

It would not be desirable to run a night shift unless absolutely necessary, if we cannot depend upon employees to be present; if we find the quality unsatisfactory; and if we note an increase in accidents.

From the standpoint of production it might be different in an unusually large plant where one machine producing three to four thousand equivalent eight inch block per shift could produce in two shifts all the block necessary for a production of 1,500,000 block, annually. But if eight inch and twelve inch block were desired frequent changing of the machine would be necessary. A machine would have to work a certain number of days or weeks on twelve inch and then change to eight inch block. A large enough stock would have to be carried to tide over the periods of non-production on each size. The cost of making changes could be fairly well determined by figuring five hours to make the change each time. Assuming twenty-four changes per year, and cost of changing to be at least one dollar per hour for 120 hours, the total would be $120.00 per year.

In case of a breakdown in a plant depending upon one machine day and night, the loss in production is considerable, especially in case of a major repair. Where two machines are used in a single shift plant and major repairs are necessary on one machine, then the other machine can take the increased load so as not to lose production.

A plant must expect breakdowns of approximately five per cent of the working time, and machines must be looked after carefully. If a machine is running day and night it cannot be given the attention that could be given to a machine running only one shift per day. The loss in production due to a breakdown in a two shift plant, especially during rush periods, might easily pay the interest on the investment in a second machine.

Depreciation per unit of production is the same whether a machine is worked single or multiple shifts. A machine has a life of a certain number of block, say one or two million, and not a life of a certain number of days or years. If a machine produces one million block in two shifts per day, it will last about one-half as long as a machine working at the same speed one shift per day.

There is also another factor to take into consideration which cannot be given a specific value, and that is the question of ex-

pansion. No one can tell when a plant is built exactly how much of a demand will be created. We may design for one million units and then find we must meet an unexpected demand for a million and a half units. In a plant for single shift operation the peak demand could be taken care of by a night shift temporarily, but in a plant designated for multiple shift operation there would be little chance to build up production to meet this increased demand, without a considerable expenditure for additional plant and equipment.

It is difficult to generalize the design of a plant, because so much depends upon local conditions and in each case a careful analysis of layout and equipment must be made to secure lowest unit cost and dependable manufacturing conditions.

From this analysis we find in the multiple shift plant:

1. Slightly lower investment.
2. Higher production cost at night due to
   (a) Bonus for night work
   (b) More breakage on night shifts
   (c) More accidents on night shifts
   (d) Need for additional supervision

In the single shift plant we find:

1. Greater flexibility in handling machines.
2. More uniform production.
3. Better quality of product.
4. Opportunity to meet unexpected demands and less machinery trouble.
5. Better supervision.
6. Lower unit cost of production.

On the basis of this comparison it is evident that in the concrete products industry the single shift plant is to be preferred over the multiple shift plant.

Readers are referred to the JOURNAL *for June 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.*

Some Tests of Concrete Masonry Units Cured with High Pressure Steam

BY P. M. WOODWORTH*

Mr. Woodworth's paper, accompanying a revival of interest in a subject actively discussed 15 to 20 years ago, raises questions which may be faced with a greater incentive to find the answers. Products men are interested in high early strengths and in taking out initial shrinkage. If high pressure steam curing will give consistent results the high cost of the equipment is less a factor in the industry than it was a decade ago. There is also the old question whether all the available cements will react favorably to this treatment. This is scheduled for convention presentation and discussion.—EDITOR

INTRODUCTION

THE RELATIVE value of different methods of curing of concrete products has long been a live question. Numerous investigations, with low pressure steam and fog spray systems, have developed certain standard practices for these two methods. Tests by Wig† and others have shown that high pressure steam curing has a decidedly beneficial effect, especially upon early strengths. The use of this method, however, has not been general in products, first, because of the comparatively high initial cost for equipment and second, because of the meagre information available as to the value of this method.

PURPOSE AND SCOPE OF TESTS

The main purpose of this investigation was to obtain data concerning the effect of high pressure steam curing on: (1) Early

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†Technological Paper No. 5, U. S. Bureau of Standards, 1912.

strength, (2) Ultimate strength, (3) Strength as affected by time interval between molding and curing.

In addition to the strength tests, shrinkage measurements were made on small masonry panels built with units cured in steam at high and low pressures.

The investigation included compression tests on 152 5 by 3¾ by 12-inch, two rectangular-core, closed-top tile, at the ages of 2, 9 and 28 days and 1 year. The 1-year specimens will be tested in April, 1930. Three specimens for each age were tested "as cured" and three specimens were immersed in water for 18 hours, capped and tested 6 hours later. The specimens were made in four groups and cured in the following manner:

Group	Method of Curing
1	*High Pressure Steam:* ½, 4, 8 and 24 hr. after molding, specimens cured for 12½ hr. in saturated steam at 100 lb. gage pressure and then stored in air until test.
2	*Steam:* 4 hr. after molding specimens cured for 12½ hr. in saturated steam at 125° F. and then air-cured until test.
3	*Air:* 6 hr. after molding specimens cured under damp straw for 24 hr. and then air cured until test.
4	*Moist.* Specimens cured under damp straw for 42 hr. Moist cured at 70° F. until test.

MATERIALS AND METHODS

The investigation was conducted at the plant of the Crume Brick Co., Dayton, Ohio. In the manufacture of tile at this plant, the molds are vibrated in order to consolidate the concrete. The plant had facilities for both low pressure and high pressure steam curing.

Portland cement from fresh warehouse stock was used.

Aggregates for the tile consisted of bank sand and crushed gravel obtained from the sidehill pit of the Crume Brick Co. The sand was graded up to the No. 8 sieve and the crushed gravel from the No. 16 sieve to ⅜ inch.

Mixing water was from the available plant supply.

The plant methods of the Crume Brick Company were used in the manufacture of the tile and were as follows:

The cement, and fine and coarse aggregate were mixed in the proportions of 1:4:6 by volume. Two-sack batches were mixed in a 14 cu. ft. Blystone mixer, 1 minute dry and 5 minutes wet. The amount of water required to produce the desired consistency was found to be equivalent to 8½ gal. per sack of cement corrected for absorption of aggregates.

A two-sack batch was sufficient to make about 170 tile or 85 tile per sack of cement. Concrete somewhat in excess of the amount required to fill the molds was then placed in a 24-gang sheet steel mold and vibrated continuously for 50 seconds. The consistency was such that after vibrating for about 20 seconds mortar appeared through the slots in the side walls of the mold. After vibrating, the excess concrete was screeded from the top of the mold and the tile lifted from the cores. The gang molds were then stacked upon industrial cars which later were run into the curing chambers.

CURING

High Pressure Steam Curing—(Group 1)—High-pressure curing was accomplished in a steel cylinder 60 ft. long and 6 ft. in diameter which was one of a battery of five regularly used in the production of sand-lime brick. The same curing regularly given the sand-lime brick was used for the concrete specimens which was as follows:

The specimens, in the gang molds on industrial cars, were wheeled into the curing chambers and the head of the cylinder bolted in place. The steam was then turned on, the air exhausted, and the pressure gradually increased to 100 pounds. This operation required about 3 hr. The pressure was maintained for 8½ hr. after which the steam was exhausted from the cylinder in one-half hour. The head was then removed and the specimens allowed to cool in the molds in the steam cylinders from 3 to 4 hr. when they were removed from the molds and stacked in the plant in such manner as to obtain free circulation of air around them until shipped.

Steam Curing—(Group 2)—The cars containing the gang molds were run into the center section of a low pressure steam tunnel 60 ft. long provided with canvas curtains or partitions 12 ft. apart. A temperature of about 125° F. was maintained for 10 hr. Gage pressure reading on the steam line averaged about 4 lb. After this treatment, the specimens were removed from the molds and stacked in the plant in the same manner as the high pressure steam-cured specimens.

Air Curing—(Group 3)—The specimens in this group were cured in the gang molds under wet straw for 24 hr. They were

removed from the molds 48 hr. after molding and air-cured in the plant until shipment to the laboratory.

All specimens in the above 3 groups to be tested at 9 and 28 days and 1 year were shipped to the Laboratory when 3 days old. They were carefully packed in straw in cardboard containers holding two tile each. The specimens were in transit four days. Upon arrival they were unpacked and stored until tested in a room where the relative humidity was maintained at 50% and the temperature at 70° F.

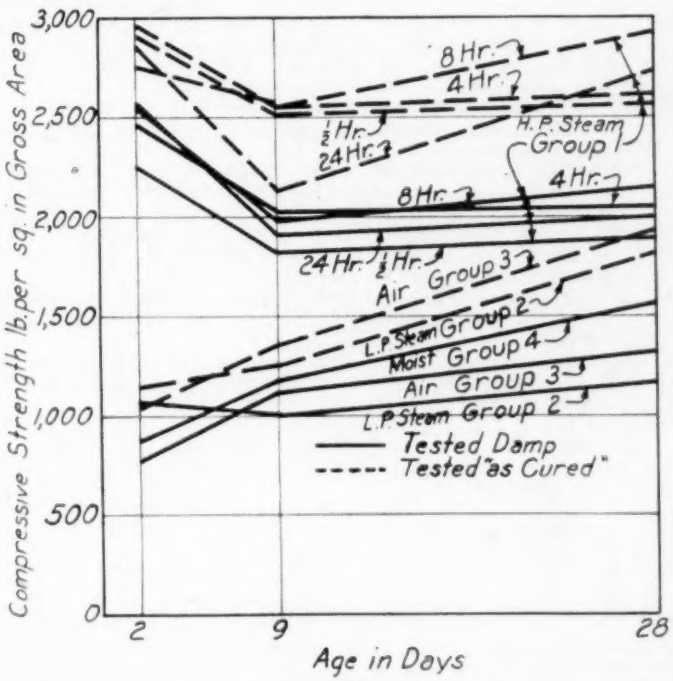

FIG. 1—EFFECT OF CURING AND CONDITION AT TEST ON COMPRESSOR STRENGTH OF VIBRATED CONCRETE TILE

Moist Curing—(Group 4)—Moist-cured specimens were kept under damp straw for 42 hr. and then removed from the molds. They were kept saturated under wet burlap at the plant until shipment to the laboratory when 3 days old. Upon arrival, at

TABLE 1—COMPRESSIVE STRENGTH OF VIBRATED CONCRETE MASONRY UNITS

Compression tests of 5 by 3¾ by 12-in. concrete tile.
Portland cement from fresh warehouse stock.
Aggregate—Crume Brick Co. bank sand and crushed gravel graded 0⅜ in.
Water—8½ gal. per sack corrected for absorption of aggregate.
Mix 1-4-6 damp, loose volume; 1-9.6 dry weight.
Concrete vibrated in molds for 50 seconds at 3600 r. p. m.
Specimens removed from molds 36 to 48 hr. after molding and cured in air until test.
Specimens capped with gypsum 6 hr. before testing.
Unless otherwise noted each value is the average of three tests made on the same day.
Values in parentheses are percentages of 28-day strengths of moist-cured tile.

Age Before Curing –Hr.–	Method of Curing	Condition When Tested	Compressive Strength Lb. Per Sq. In. Gross Area					
			2 Da.*		9 Da.		28 Da.	
½	12½ Hr. in saturated steam at 100 lb. gage pressure then air-cured at 70° F.	Damp	2250	(144)	1820	(117)	1890	(121)
		As Cured	2900	(186)	2500	(160)	2550	(164)
4	12½ Hr. in saturated steam at 100 lb. gage pressure then air-cured at 70° F.	Damp	2440	(157)	2020	(131)	2060	(132)
		As Cured	2960	(190)	2560	(164)	2620	(168)
8	12½ Hr. in saturated steam at 100 lb. gage pressure then air-cured at 70° F.	Damp	2530	(161)	1970	(126)	2150	(138)
		As Cured	2760	(177)	2580	(165)	2990	(192)
24	12½ Hr. in saturated steam at 100 lb. gage pressure then air-cured at 70° F.	Damp	2560	(164)	1880	(121)	1990	(128)
		As Cured	2840	(182)	2140	(137)	2740	(176)
4	12½ Hr. in steam at 125° F. and atmospheric pressure then air-cured at 70° F.	Damp	1070	(69)	1000	(64)	1190	(76)
		As Cured	1150	(74)	1270	(81)	1810	(116)
6	12½ Hr. under damp straw at plant temperature then air-cured at 70° F.	Damp	770	(49)	1110	(71)	1310	(84)
		As Cured	1060	(68)	1360	(87)	1930	(124)
0	48 Hr. under damp straw at plant temperature then moist-cured at 70° F.	Damp	860	(55)	1190	(76)	1560	(100)

*2-Day Tests made on 200,000 lb. Testing Machine at Bowser-Morner Testing Laboratories, Dayton, Ohio.
Tests at later ages made on 300,000-lb. Testing Machine at Research Laboratory of Portland Cement Association, Chicago.

age of 7 days, they were stored in a moist room at a temperature of 70° F. and a relative humidity of approximately 100%.

METHOD OF TESTING

Moist-cured specimens were tested damp; for the other 3 methods duplicate sets of specimens were made, 1 set being tested "as cured" (in a room-dry condition) and the other set after soaking in water at 70° F. for 18 hr. The 2-day specimens

were tested in a 200,000-lb. hand-operated hydraulic testing machine at the Bowser-Morner Testing Laboratories, Dayton, Ohio. Compression tests at the other ages were made in a 300,000-lb. testing machine at the Research Laboratory of the Portland Cement Association, Chicago. All specimens were capped with plaster of paris on both bearing surfaces the day of test and were loaded through a spherical bearing block placed on top of the specimens.

DATA AND DISCUSSION OF TESTS

Table 1 gives results of compression tests on the tile for the different methods of curing at ages of 2 days to 28 days, and

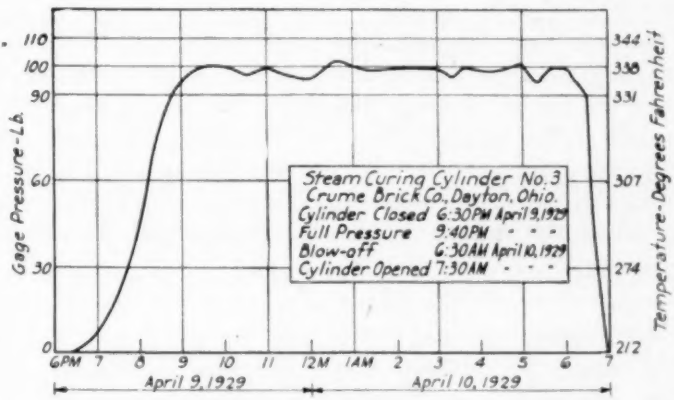

FIG. 2—PRESSURE GAGE CHART FOR HIGH PRESSURE STEAM CURING CHAMBER.

strength ratios based on tests of the 28-day moist-cured specimens used as the basis of comparison. The principal strength relations for the different curing methods are shown in Fig. 1. Pressure gage readings are shown in Fig. 2. The average temperature outdoors at the plant during the making of specimens was about 48° F.

Age-Strength Relations—The curves in Fig. 1 show the compressive strengths of the tile at different ages. By means of these curves the various methods may be readily compared. In general high pressure steam curing had a decidedly beneficial effect on the strength at all ages.

Early Strength—The 2-day strengths of the tile which were steam cured for 12½ hr. at 100-lb. pressure were about 55% higher than the 28-day strengths of the moist-cured tile, and were more than double the 2-day strengths of tile cured in low pressure steam. These comparisons are on the basis of specimens tested in a damp condition.

28-Day Strength—The strengths of the high pressure steam cured tile show a general downward trend between the ages of 2 and 9 days followed by an increase in strength between the ages of 9 and 28 days. The reason for the falling off in strength at 9 days for each of the different groups of specimens cured with high pressure steam is not apparent and this point should be given further study.

Effect of Time Interval Between Molding and Curing—The curves in Fig. 1 show that the time elapsing between molding and placing the tile in the high pressure steam curing chamber did not have an appreciable effect on compressive strength except in the case of the units placed in the curing chamber ½ hour after molding. In this case the strengths averaged about 90% of those obtained for the other intervals which ranged from 4 to 24 hr.

Other Curing Methods—The strengths of low pressure steam cured and air-cured tile tested in a room-dry condition were generally higher than the moist-cured tile tested wet, but when tested damp they showed about the same or slightly lower strengths.

Shrinkage Studies—Supplementing the strength tests, shrinkage measurements were made on 3 panels laid up with units cured in high pressure and low pressure steam. In these studies the following methods were used:

Masonry panels one unit wide, two units deep and thirteen units high were laid up using a ⅜-in. mortar joint of 1-1-6 cement-lime mortar. The dimensions of the finished panels were 12 by 8 by 70 in. Thin metal ties were used as a bond between the two individual walls of the panel. Gage points were placed near each of the 4 corners so to establish four gage lines over which shrinkage measurements could be made. The gage length included 12 units and eleven mortar joints and was approximately 64 inches. Readings were made with a 1/1000-in. Ames dial mounted on a

5/8-in. pointed steel rod. These readings were made immediately after erection of the panels and continued for a period of 230 days.

Following are details of the three panels:

Panel No. 1—High pressure steam-cured units laid up in a room-dry condition at the age of 15 days.

Panel No. 2—High pressure steam-cured units immersed in water for 24 hr. with subsequent air drying for 6 hr. previous to laying up in the column. This simulated the worst condition

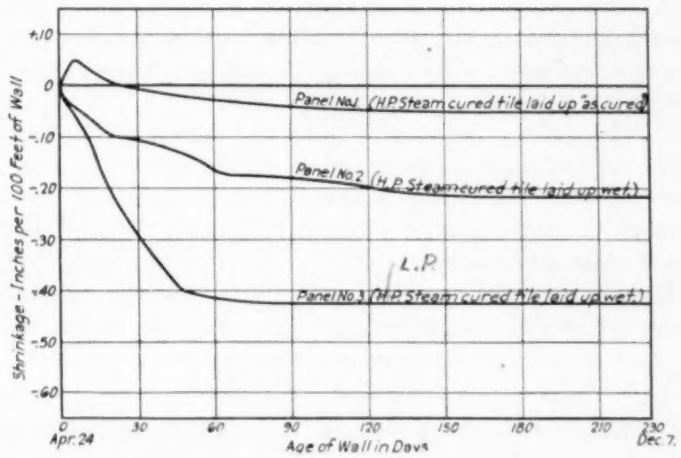

FIG. 3—SHRINKAGE OF CONCRETE MASONRY PANELS.

likely to occur in the field so far as moisture content of units is concerned. The units were 15 days old when laid up in the panel and contained 5.0% moisture by weight.

Panel No. 3—Low pressure, steam-cured units were laid up when 15 days old after soaking in the same manner as the units in panel No. 2. Their moisture content was 5.8% by weight.

A study of the curves in Fig. 3 shows that the maximum change for Panel No. 1 was practically .05 in. per 100 lineal ft. of wall. Panel No. 2 had a maximum shrinkage of approximately .22 in. per 100 ft. of length, which was about one-half that (.45 in. per 100 ft.) obtained with Panel No. 3 in which low pressure steam-

cured units were used. The measurements on these three panels clearly indicate that high pressure steam curing greatly minimized the shrinkage of the panels and where air-dry units were used (Panel 1) the shrinkage was practically nil.

SUMMARY

The more important indications from the results of the tests are as follows:

(1) Units cured in high pressure steam developed strength at two days greatly in excess of the strength obtained at 28 days by moist curing, air curing or low pressure steam curing.

(2) Although there was an apparent retrogression in strength of the high pressure steam-cured units between the ages of 2 and 9 days, there was a slight increase in strength between the ages of 9 and 28 days in all cases.

(3) The interval between molding and curing in high pressure steam did not have an appreciable effect upon the compressive strength of tile steamed 4, 8 and 24 hr. after molding but units steamed one-half hr. after molding had about 90% of that obtained for longer intervals.

(4) High pressure steam curing greatly reduced shrinkage of concrete masonry panels laid up with wet units and when air-dry units were used the shrinkage was practically nil.

CONCLUDING REMARKS

It is believed that high pressure steam curing has possibilities as a means for developing concrete products having high early strength and low shrinkage when laid up in masonry walls. The results of these tests should not be construed as conclusive. Additional information concerning the effect of high pressure steam curing on strength of units shrinkage of masonry walls using different aggregates, methods of molding, consistencies, gradings, durations and variations of pressure should be obtained before definite conclusions concerning the value of this method of curing concrete products can be drawn.

Readers are referred to the Journal for June, 1930 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1930.

Abstracts

of the current literature of cement and concrete

Prepared under the Direction of the
American Concrete Institute
Committee on Abstracts

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MATERIALS

Methods of inspection of mineral aggregates. A. S. RAE. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 745-753.—This paper discusses the methods of inspection of mineral aggregates from a technical standpoint. The discussion considers: (1) preliminary investigations, including material surveys and reconnaissance surveys: (2) inspection during construction, including inspection at source of supply and at destination; (3) methods of sampling; and (4) field tests. The important things to be observed in inspection at the source of supply and at destination are pointed out and the advantages of each method of inspection are discussed. The importance of proper sampling in emphasized and the standard methods of the Society are recommended as a guide in formulating instructions to inspectors. The screen test, silt test, colorimetric test for sand and test for weight per cubic foot are named as the more important field tests. Mineral aggregates play a most important part in the construction work throughout the country. Our highways and streets, for example, are over 90 per cent mineral aggregates and concrete construction of all kinds will average 85 per cent aggregates. Therefore adequate inspection and tests are necessary to the protection of the enormous annual investment in these materials.—(From synopsis by Am. Soc. Testing Materials.)

Phoenix Utility Co., furnished mixed concrete for hydro-electric development. F. C. LINCOLN. *Pit and Quarry*, Dec. 4, 1929, V. 19, p. 63-67.—Plant was erected solely to provide concrete in the construction of the Morony dam across Missouri River 17 miles below Great Falls, Mont. Pouring of concrete began inside of four months from time ground was broken. Dam is 850 ft. long and 95 ft. above river-bed. Cement is received in bulk in box-cars and is unloaded by a power shovel into an inclined silo built on the slope of the hill. It feeds from this silo by a screw conveyor to an 18-in. belt conveyor, to a bin over the concrete-mixing plant. Pit-run gravel is brought 40 miles in bottom-dump cars. Before shipment all stone over 4-in. is removed. At dam plant the aggregate is thoroughly washed and screened into following grades: 4-in. to 1-in.; 1-in. to ¼-in.; sand. Concrete contains 1.15 bbl. cement per cu. yd. Aggregates are proportioned by weighing. Mixing is done in two 2-cu-yd. machines. Each batch is composed of 6,850 lb. of aggregate, 865 lb. of cement, and 50 gal. water, and is mixed for two minutes.—A. J. HOSKIN

The Cement shall. R. R. COGHLAN. *Concrete* (M. S.), Nov., 1929, V. 35, No. 5, p. 101-104.—The author reviews the methods of test and points out many places where the failure rigidly to observe the prescribed technique results in marked departures in result from those which would obtain by correct test. Of special significance are the temperature and humidity conditions of the moist closet. The design is given for a moist closet which will meet the desired requirements.

Studies on fundamental synthesis of calcium aluminates and their hydration. S. NAGAI and R. NAITO. *Concrete* (M. S.), Nov. 1929, V. 35, No. 5, p. 119.—The authors studied the formation of $CaO.Al_2O_3$ and $5CaO.3Al_2O_3$ by heating mixtures of Al_2O_3 and $CaCO_3$ in the proportions of 1 to 1 and 5 to 3. Free CaO by the ammonium acetate method and free Al_2O_3 by insoluble residue were determined. The ratio of combined CaO to combined Al_2O_3, obtained by difference, is compared with the theoretical ratios, 1 to 1.67. The ratios obtained with the 1 to 1 mixture, heated at temperatures ranging from 950° to 1250° fall between 0.47 and 1.21. With the 5 to 3 mixture, the ratios obtained at the same temperature fall between 0.52 and 1.64. The ratios, together with specific gravities of the products, are used to calculate the amounts of $CaO.Al_2O_3$ and $5CaO.3Al_2O_3$ formed.

Sand and gravel, and central-mixing plants combined by South Bend company. *Pit and Quarry*, Dec. 4, 1929, V. 19, No. 5, p. 71-76.—Describes operation of Geo. J. Hoffman & Co. plant which has a capacity of 600 cu. yd. per day. Describes pit

operations, screening and washing into five sizes of aggregate. Cement is purchased in bulk, in box-cars, is unloaded by an automatic scraper into a screw conveyor, and fed to bucket elevator which stores it in a bin. If this bin be already full, cement is spouted into a 900-bbl. storage house whence it can be recovered by same screw conveyor and elevator. Each of the two mixers is provided with three batchers—for sand, gravel, cement—each batcher having its own hopper scale, a few seconds being required to proportion each batch. Water is measured in special tanks with gauge-glass. Batchmeters control each batch to a 2-min. mix. One mixer is 2-cu. yd., the other 1-cu. yd. capacity. Concrete is hauled in trucks, as far as 16 miles.—A. J. HOSKIN

Fine aggregate in mortar and plaster. J. C. PEARSON. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 774-787.—This paper presents a brief discussion of the functions and properties of ordinary mortar and plaster, and shows the relatively minor and more or less indeterminate contribution of the fine aggregate to these properties. Hence the existing lack of attention to quality of fine aggregate for mortar and plaster, Existing specifications for lime and gypsum plastering sands are not entirely satisfactory, as they undoubtedly debar many suitable aggregates. Data most needed for a logical selection of aggregate have to do with plasticity and volume change, but very few have been secured up to this time and more fundamental studies are needed to supply such data. The application of the so-called plastic mortar test to 35 building sands indicates that this is a safeguard against dangerous sands, and that poorly graded or excessively fine sands are uneconomical. However, a complete and satisfactory specification for fine aggregate can hardly be written until general relations are established between the characteristics of fine aggregates and the properties of plasticity and volume change in mortars and plaster.—(From synopsis by Am. Soc. Testing Materials.)

Stone screenings. *Rock Products*, Dec. 21, 1929. V. 32, No. 26, p. 106.—Uses listed in summary of aritcle by A. T. GOLDBECK in *Crushed Stone Journal* includes: (1) Fine aggregate in concrete, alone or in combination with sand; (2) binder in waterbound macadam road construction; (3) portion of the aggregate in bituminous concrete; (4) cover material in surface treatment for roads; (5) subgrade treatment under macadam, concrete and other types of roads; (6) low cost road construction; (7) shoulder maintenance; (8) concrete block manufacture; (9) miscellaneous uses in concrete.—J. J. LANDY

Influence of different kinds of sand on the mechanical properties of cement. P. P. BUDNIKOFF. *Zement* (Germany) Dec. 1929, V. 18, No. 49, p. 1410-1.—Experiments were made with several portland cements and different kinds of sand in order to find a sand suitable as a standard sand for physical tests. The sand from Wolsk (Russia) has these properties. Microphotographs are given.—A. E. BEITLICH

"Si-Stoff" and trass as aggregates for cement and lime. R. MEUSER. *Zement* (Germany) Dec. 1929, V. 18, No. 49, p. 1411-5.—The use of "Si-Stoff" for the construction of dams can not be recommended; trass is a much better aggregate. The amount of soluble silica is higher in trass, than it is in "Si-Stoff." Trass has a better plasticity. The compressive strength of "Si-Stoff" is slightly higher but at the cost of a lower density. Concrete with trass as aggregate shows its highest strength after a long time period, while "Si-Stoff" gives an earlier maximum in strength. Dr. Luftschitz disputes this conception and claims, that "Si-Stoff" is a better aggregate than trass for the construction of dams. (cf. Dr. Luftschitz, *Zement* 1929, V. 18, No. 39.)—A. E. BEITLICH

The relation between the fineness of grinding and the retrogression of strength of portland cement. N. NICOLAESCO. *Rev. materiaux construction trav. publics.* (France) 1929, p. 401-2.—Cement exposed to damp air losses in strength more when finely ground than when less finely ground; but when out of contact with the air the strength is not affected regardless of the fineness —F. O. ANDEREGG.

Rapid determination of calcium oxide in portland cement, C. H. ROGOZINSKI. *Zement* (Germany) Dec. 1929, V. 18, No. 51, p. 1464.—To 0.65 g. of cement add 5 cc. of water and 20 cc. N-HCl, break up all the lumps and let stand for 15 min. on a warm hot-plate. Dilute with boiling water to 200 cc., boil for 1 min. and titrate hot with N/2 NaOH-solution and phenolphthalein as indicator. A filter-paper behind the beaker shows the first appearance of a pink color. The accuracy of the method is 0.1 per cent calcium oxide. It is advised to run duplicates.—A. E. BEITLICH

The effect of zinc and zinc oxide on the initial set of cements with a slag base. EMMANUEL PUJOL. *Rev. materiaux construction trav. publics.* (France) 1929, p. 407-9.—Zn and ZnO even in traces have appreciable effect in slowing down the initial set of cements with a slag base. Therefore, a zinc table top should not be used for making up test specimens. The action is similar to that of PbO.—F. O. ANDEREGG

The rate of hydration of cement clinker. F. O. ANDEREGG and D. S. HUBBELL, *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 554-564.—Lack of knowledge of the rate of hydration of cement clinker has hindered a more complete understanding of the complicated setting and hardening reactions that cements undergo. Carefully sized fractions have been separated by prolonged elutriation with dried air and gaged with water under a variety of conditions. After varying periods of storage, samples were removed and rapidly dried. The partially hydrated material was ground in an agate mortar until a high degree of uniformity of size was secured. The ground powder was made into a slide and the ratio of hydrated to unhydrated determined. Storage in water at 70° F. resulting in the hydration of a layer of a typical portland-cement clinker to an average depth of about one-half micron in 24 hours, 1.7 in 7 days, 3.5 in 28 and about five microns in 90 days, in the presence of calcium sulfate. In the absence of calcium sulfate, the penetration appears to be somewhat greater. Calcium chloride in the presence of calcium sulfate does not seem to have much effect on this reaction. A temperature of 32° F. slowed down the initial reaction and one of 100° F. increased it at early periods, but no appreciable difference could be detected at 28 and 90 days at either temperature from similar hydrating cement stored at 70° F. One portland cement of high early strength was apparently hydrated more completely at 7 and 28 days than ordinary cement. The greater apparent hydration of a white portland cement is explained by the larger surface area due to greater irregularity in shape.—(From synopsis by Am. Soc. Testing Materials.)

Fine aggregate in concrete. H. F. GONNERMAN. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 754-773.—This paper gives a general discussion of the role of fine aggregate in concrete, and includes brief statements from the published writings of some of the early writers on the subject, who were concerned with many of the problems that are discussed in current engineering literature. The function of fine aggregates in concrete and the principal considerations involved in their selection are discussed in detail. The desirable characteristics of a sand for use in concrete are stressed, including durability and cleanness of the particles and tests for these properties are reviewed. The use of crushed stone or air-cooled slag screenings as fine aggregate is also discussed. The influence of size and grading of fine aggregate on such properties as strength, permeability, and workability of concrete is pointed out, and the principal requirements of typical specifications for fine aggregate summarized.—(From synopsis by Am. Soc. Testing Materials.)

Sugar in sand: a quick test to determine its presence bacteriologically. N. H. ROY. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 611-615.—The effects of sugar in portland-cement mortar and concrete have been discussed by many authors, and the serious and costly effects of its presence are so commonly

known that a discussion of them is not necessary in this paper. Sugar has been found in sand from some deposits, and doubtless there have been many others in which its presence was unknown and unsuspected. The presence of sugars in natural sands may account for many cases of concrete that will not set or harden or that have low strength. As the effects of even small amounts of sugar generally are to ruin concrete, a method for detecting its presence in sand is discussed in this paper, together with a statement of the classes of sugars found naturally in fields and streams, and of the bacterial action that takes place in nature. The proposed test is the reversal of a common bacterial test —the use of a common harmless bacterium, Aerobacter Aerogenes (Bacterium Aerogenes), to test for the presence of sugar. A washing from the sand is placed in a test tube together with a small quantity of bouillon, the mixture is inoculated with Aerobacter Aerogenes and set aside in a warm place for a short time. The presence of sugar is revealed by the liberation of carbon dioxide which is easily collected in a fermentation tube to furnish visual quantitative evidence. Carbon dioxide will not be liberated by the bacteria from organic materials such as tannic acid. The method is rapid and reliable and will detect very minute quantities of sugar. Supporting data and photograph showing the results of tests are included.—(From synopsis by Am .Soc. Testing Materials.)

Winter thoughts of an aggregate salesman. S. M. HANDS. *Rock Products*, Dec. 7, 1929, V. 32, No. 25, p. 52-60.—Data are furnished to show that good concrete can be made from clean crusher run or pit run materials. Standard grading only practical and effective when they give uniform quality of concrete. Author suggests engineer study materials and base design to make fool-proof mixtures. Workability of concrete is essential and best—all designs should aim at this. Grading requirements should not be too rigid but allow for small changes due to segregation—uniform grading is generally sufficient. Any specifications which increase costs of aggregate manufacture increase concrete costs, hence the prime requirement is workability. Slump curve should be flat through the range of consistencies required for the work, thereby indicating serious grading changes or water increases. All proportioning should be by weight, thus simplifying control with definite limit for water. Regarding sand, decision as to amount should be decided by engineer with workability as prime consideration. Too much stress is put on "cement saving" aggregates which is reflected in higher cost of resulting concrete, because of increased prices paid for them.—J. J. LANDY

Tricalciumsilicate. ERNST JAENICKE. *Zement* (Germany) Dec. 1929, V. 18, No. 48, p. 1388-9.—Dispute between E. Jaenicke and A. Guttmann and F. Gille about the existence of the tricalciumsilicate. (cf. Jaenicke. *Zement* 1929, V. 18, p. 1345-7.)—A. E. BEITLICH

Comment on Walker's analysis of Bureau of Public Roads tests. EDMUND SHAW. *Rock Products*, Dec. 21, 1929, V. 32, No. 26.—Shaw upholds Walker's comparative figures stating that the results are more equitable because the effect of aggregates on cement only are compared and not the adaptability of aggregates to some particular mixture. The works of Gray, Kendall and Foster are cited.—J. J. LANDY

Sand and gravel prospects in specific local area. FREEMAN WARD. *Rock Products*, Dec. 7, 1929, V. 32, No. 25, p. 68-71.—Uses, markets and definitions of various grades of sand and gravel in Reading, Penn., area given. Favorable possibilities found along the Schuylkill river near Pottstown, although size and extent of deposits have yet to be determined. The sources of sand and gravel in the district are fully investigated.—J. J. LANDY

Portland Cement Association plans long series of safety meetings for 1930. *Rock Products*, Dec. 7, 1929. V. 32, No. 25, p. 83-86.—Fifteen regional conferences are planned to reach almost all sections in the United States.—J. J. LANDY

A compression test for portland cement. O. L. MOORE. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 570-584.—It is generally known that standard Ottawa sand mortar of plastic consistency gives test results considerably lower in compression than workable concrete of the same water-cement ratio. Otherwise standard sand is ideal for standardization purposes in that it is practically constant in grading, size and structure, does not absorb water and is established as the standard testing sand. This paper offers a practical method, developed in the Central Laboratory of the Universal Portland Cement Co., whereby the compressive strength of standard sand mortar of plastic consistency is a satisfactory measure of the strength property of portland cement in concrete. Data are given to show that the compressive strength of plastic standard sand mortar is a function of its voids-cement ratio; that if the cement factor is expressed in the same terms in voids-cement ratio of standard sand mortar and in water-cement ratio of concrete, then the strengths are approximately equal when the two ratios are equal; that by means of a straight-line chart or equation between voids-cement ratio and water-cement ratio of standard sand mortar, the strength of the latter becomes a direct measure of concrete strength on a water-cement ratio basis. An example illustrating the method of procedure and data showing results obtained with fifty different brands of portland cement are given. A method for testing sands and a basis for predetermining compressive strength are suggested. The method of test offers a good criterion for portland cement strength in concrete. It utilizes a consistency which is subject to minimum error in mixing and molding, and it has interesting possibilities for standardization because of the uniform properties of standard sand.—(From synopsis by Am. Soc. Testing Materials.)

Influence of mineral composition of aggregates on fire resistance of concrete. (See PROPERTIES OF CONCRETE.)

Influence of quality of coarse aggregate on strength of concrete. (See PROPERTIES OF CONCRETE.)

Influence of coarse aggregate on the durability of concrete. (See PROPERTIES OF CONCRETE.)

Dragline excavators. J. C. FRENCH. *Rock Products*. Dec. 7, 1929, V. 32, No. 25, p. 106-107.—Description of various types of machines and their relative value in stripping, excavation and general operations.—J. J. LANDY

Waste heat recovery at Security, Md., plant of North American Cement Corp. *Rock Products*, Dec. 21, 1929, V. 32, No. 26, p. 53-58.—Description of new generating plant producing 6000 kw. per day from waste kiln gases. Advantage was taken of previous experience and various refinements not incorporated in earlier plants were included in design and construction.—J. J. LANDY

Diesel engines in the rock products industries. ORVILLE ADAMS. *Rock Products*, Dec. 21, 1929, V. 32. No. 26, p. 59-67.—Part 4 of series. Development and applications of Diesel engines to portable equipment as draglines, shovels and locomotives discussed. Some comparative cost figures are given.—J. J. LANDY

Preparing clean stone from a former waste product. *Rock Products*, Dec. 21, 1929, V. 32, No. 26, p. 68-69.—At Bethlehem Mines Corp. stone washing plant, a former waste product is converted to marketable material by washing and screening. Operation details are given.—J. J. LANDY

Spray process of slurry feed. S. J. M. AULD. *Rock Products*, Dec. 21, 1929, V. 32, No. 26, p. 70-72.—Descriptive details concerned with installation of Rigby cement slurry spraying system in an English cement mill. Installation costs, data on fuel consumption and power are given. *cf. Cement and Cement Manufacture*, Nov. 1929.—J. J. LANDY

Report on research results. R. NACKEN. *Zement* (Germany) Dec. 1929, V. 18, No. 47 and 48, p. 1366-70, 1389-92.—(1) A crystalline product was

found in a special clinker and its properties were studied by chemical and microscopical examinations. The analysis showed a compound of the type $2CaO.Al_2O_3.SiO_2$. The crystals are tetragonal, the refractive indices are $\alpha = 1.657$ and $\delta = 1.667$. These optical properties agree with the synthetic $2CaO.Al_2O_3.SiO_2$. (2) Solubility determinations were made with clinker, cement and gypsum in water in order to study the hydration of portland cement. The formation of a protecting colloidal film around the particles is assumed. (3) Studies of the heat tones of the reactions of clinker and cement with water. Clinker combines with 3 per cent and cement with 1 per cent water during the first 30 min., then a standstill of the reaction takes place until a second reaction shows a continuation of the combination with water. (4) Studies of the influence of LiCl, NaCl and KCl on the setting of portland cement. The heat of reaction and the combination with water increases with the molecular weight of the alkali chloride. Solutions of $CaCl_2$ of different concentrations show, that the heat tone is greater in the case of the stronger solutions (4N and 6N) during the first two hours, but it drops below the heat tones of the weaker solutions (2/5N and 1/5N) after ten hours. $BaCl_2$-solutions and HCl show a similar behavior. H_2SO_4 and HNO_3 retard the combination with water, NaOH accelerates it. $Ba(OH)_2$ and $Ca(OH)_2$ have little influence. In every case two different reactions take place. The results are compared with determinations of the electrical conductivity of solutions of cement in water.—A. E. BEITLICH

A method for a rapid determination of dust in air. WALTER TRAUTE. *Zement* (Germany) Dec. 1929, V. 18, No. 51, p. 1474-5.—A certain amount of air is sucked through a Gooch-crucible which is filled with 5 g. of fine pulverized sugar. After dissolving the sugar in 20 cc. of water the solution is centrifuged for two min. at a speed of 2000 r. p. m. in a test tube, which is 18 cm. long and 2 cm. in diameter. On the end of this tube is sealed a capillary tube, 3 cm. long and 1 mm. inside diameter. A small amount (0.01 g.) of minium is centrifuged to the bottom of the capillary tube in order to obtain a level surface for the dust sample. The tube must be standardized with weighed amounts of dust. The determination of the dust is made by measuring the length of the centrifuged dust column. Accuracy of 0.2 mg.—A. E. BEITLICH

Combinations of silica and lime. ANON. *Rev. materiaux construction trav. publics.* (France) 1929, p. 429-22.—A general description of the making of sand-lime brick.—F. O. ANDEREGG

Recovery of waste heat in the cement mill. P. LEBRUN. *Rev. materiaux construction trav. publics.* (France) 1929, p. 416-9.—By filtering the slurry and using waste heat boilers an appreciable saving has been obtained.—F. O. ANDEREGG.

A new sack and a new method of sacking. J. PROUTEAU. *Rev. materiaux construction trav. publics.* (France) 1929, p. 414-6.—Description of the "Jar" sack and the "Haver" machine. —F. O. ANDEREGG

The constitution of portland cement clinker. HANS KUHL. *Tonind. Ztg.* (Germany) 1929, V. 53, p. 1571-5.—The problem of hardening and its significance in cement study. Hans Kühl. *Ibid.* V. 53, p. 1672-5, 1690-2. Lectures.—F. O. ANDEREGG

The effect of zinc and zinc oxide on the setting of various hydraulic cements. E. RENGADE. *Rev. materiaux construction trav. publics.* (France) 1929, p. 409-11.—Zn and ZnO affect several different kinds of cement adversely, increasing greatly the period of initial set and lowering the strengths considerably. The fused aluminous cement is not hurt by as much as 1 per cent ZnO.—F. O. ANDEREGG

Pacific Coast Cement Co. mill at Seattle, Wash. S. E. HUTTON, *Rock Products*, Dec. 7, 1929, V. 32, No. 25, p. 33-51.—Modern new mill of 1,250,000 bbl. annual capacity, two kilns, 11 ft. 3 in. by 240 ft., wet-process plant. Complete description, fully illustrated with detailed plans of different mill buildings.—J. J. LANDY

Atlas Portland Cement Co. opens new plant near Waco, Texas. T. K. KNOX. *Pit and Quarry*, Dec. 4, 1929, V. 19, No. 5, p. 30-36.—New, dry-process cement plant has a daily capacity of 2,400 bbl. Its first train-load of product was shipped exactly nine months from day that construction started. Both types of the raw materials are quarried in neighboring workings. Electric power operates drills and shovels in quarrying Austin chalk and Eagle Ford shale. Woodford haulage (remote-control electric) employed, using 10-cu. yd. cars. Quarry rock dumped first into steel-lined hopper, thence over reciprocating table feeder to large hammermill crusher which is only crusher in the plant, it working alternately on limestone and shale. Crushed material is taken by rubber-belt conveyor to by-pass conveyor. This last conveyor takes the limerock to a bin 32 ft. diam. by 35 ft. height. The shale is delivered directly to dryers. From the bin, the "rock" is fed, by a table feeder, to a belt conveyor, thence to a hopper, whence it is lifted by a vertical elevator to the hopper of the dryers. Two dryers, each 6 ft. by 80 ft. utilize waste gases from the kilns, but also burn gas when no kiln is operating, air for the gas combustion being furnished by electric-driven blower. Gases from dryers traverse a vortex-type dust collector, as does dust collected at all other sources in the plant. Dried material is taken by drag-chain conveyor and bucket elevator above a battery of eight storage silos, with three interstice bins. Two silos are used for dried limerock, two silos and the interstices for dried shale, and remaining four silos store ground materials. Material is delivered by belt conveyors and vertical bucket elevator to hoppers of the raw grinding mills. From these hoppers the two materials are fed to the mixing scales, the proper proportions being made before the grinding—a distinct feature of this plant. After grinding the material is taken by pneumatic system to the four silos each of which is divided into four compartments, thus affording storage for sixteen compositions. Materials are withdrawn by a time-control device which automatically operates the valves in sequence to effect a fine proportioning for the kiln-feed tank to which the ground material is conveyed by a pneumatic system. Mixture is fed to the kiln by screw conveyor which accurately regulates the feed. The kiln, 11 ft. by 250 ft., with four ring bearings, utilizes gas for fuel. Waste gas passes through chambers whence some is used for drying, the balance going to the stack. Clinker drops through a vertical cooler to a horizontal cooler set at right-angles to kiln. After being automatically weighed the clinker drops into a pit whence it is distributed, over the clinker storage. One crane-way handles the clinker and the gypsum. Finish grinding is in the same building with the raw grinding. There are eight cement-storage silos, each 26 ft. in diam. by 60 ft. in height, with three interstices. Portable pneumatic pumps transfer finished cement to packing house, or from one silo to another to attain any further desirable blending. Two automatic bag packers load to the railroad cars, one to trucks. —A. J. HOSKIN

PROPERTIES OF CONCRETE

The attainment of durability in concrete structures. JOHN H. HOUGH. *The Eng. J.*, Dec. 1929, V. 12, pp. 617-625.—Although available data do not always lend themselves readily to the establishment of a direct relation between the water-cement ratio and some required degree of durability, nevertheless this relation exists. Conforming to the water-cement ratio law, mixes may be designed not only for strength but for actual conditions of exposure. Destructive agents may be chemical or physical. If chemical, the action proceeds by the dissolving out of essential ingredients of the hardened cement paste, and the greatest deterioration occurs from penetration, which progressively increases the area of attack. If the agent be physical, as in the case of frost action on the entrained water, or through the deposition near the surface of dissolved salts as the liquid is brought to the surface and evaporated, the action is again caused by penetration. Thus, in most cases,

the greatest deterioration in exposed concrete comes from the penetration of moisture to the interior of the mass. The problem, then, is to control the factors affecting liquid or watertightness of the cement-water paste through which deleterious substances must find their way. Watertightness in a large degree determines durability. Concrete may be usefully regarded as an aggregate mass held together by an active element consisting of a hardened paste of portland cement and water. If watertight concrete is required, this paste may be watertight. The properties of the hardened paste depend on (1) the relative proportions of cement and water and (2) the extent to which the chemical combination between the cement and water is allowed to proceed. Merely sufficient water to complete hydration of the cement will produce a paste not plastic enough to be handled, but water in excess of the ideal amount for hydration serves to dilute the mixture and ultimately, through withdrawal of the excess, to produce voids and lessen the watertightness and durability. Lack of plasticity, resulting in honeycombing, greatly increases the area presented to the attack of weathering or other destructive agencies, and hence mixes must not be too stiff. Overwet mixes conduce to segregation of materials with accompanying honeycombing. Accumulation of water and fine materials at the surface produces laitance, which is itself pervious, as is the concrete immediately below laitance seams. Uncombined water is a source of porosity, and since the richer mixes have less uncombined than the leaner mixes, the former are favored. As the size of the individual pore spaces required by the uncombined water affects permeability, a rich mix with its smaller total volume of pores distributed through the larger mass of paste, will have an advantage over a lean mix with a larger number of pores distributed through a smaller paste volume. Curing, which is a continuance of the chemical reactions, involves the entering of water into combination with the cement, and is beneficial through the reduction of uncombined water. Adequate curing affects the development of strength, watertightness and other qualities governing durability more than any other single factor except the water-cement ratio. Such impermeability data as are available show that watertightness of concrete is directly affected by the water-cement ratio. Withey and Wiepking, in tests made at the University of Wisconsin, found that for properly placed and cured broken stone or gravel concrete, the permeability decreased as the compressive strength increased until the latter reached 2500 lb. per sq. in., when the concretes became watertight. Tests made by the Department of Industrial and Scientific Research, London, Eng., show that there is a relation between permeability and quantity of mixing water. For a 1:2:4 mix the permeability was least for about 6.7 U. S. gals. of water per 94-lb. sack of cement. In selecting the water-cement ratio for durability, the performance of similar structures must be considered in the light of available data on freezing, thawing and permeability. Recommended quantities of water per sack of cement are listed, having regard to various degrees of exposure, the concrete being assumed as protected from the loss of moisture for at least ten days at a temperature of 70° F. Changing either the water-cement ratio or the degree of curing does not alter the importance of the other. Within well defined limits, a particular strength or watertightness can be developed by several different water-cement ratios, provided the concrete in each case is given the appropriate degree of curing. —C. R. YOUNG

Influence of mineral composition of aggregates on fire resistance of concrete. S. H. INGBERG. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 824-829.—Fires and fire tests have shown a marked difference in fire effects on concrete made with different kinds of aggregates, which have been shown to be caused by differences in mineral composition of the aggregates involved. Four general groups are recognizable as it concerns such effects: (1) calcareous, (2) felds-

pathic, (3) granites and sandstones, and (4) siliceous aggregates. The mineral composition of the coarse aggregates has a greater influence on the fire-resistive properties than that of the fine aggregates, due to the greater amount entering into the mix and possibly to the coarser gradation. For a considerable range in fire conditions in buildings concrete can be made of any otherwise acceptable aggregates irrespective of mineral composition without danger of collapse of structural members from a fire completely consuming combustible contents and trim. For more severe fire exposures, the fire resistance of concrete subject to severe fire effects can be materially increased by placing metal ties or mesh in the outer portions of the concrete members or structural protections. Further improvement is obtainable by applying plaster or similar protections outside of the structural concrete. Concretes made of aggregates in groups (1) and (2) will generally require no special provisions in their application. There is need for further work on the effect of the fine aggregate on fire resistance, the fire resistance of concrete made with mixtures of aggregates of different mineral compositions, effect of impurities such as of silica in limestone, and the development of a standard test for fire resistance that will classify concrete with respect to fire resistance without reference to the type of structural member in which it is applied.—From synopsis by Am. Soc. Testing Materials.

Influence of coarse aggregate on the durability of concrete. F. R. McMILLAN and GEORGE W. WARD. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 816-823.—A brief review is presented of the literature bearing on failure of aggregate of various types. Igneous, sedimentary and certain of the metamorphic rocks are discussed. Consideration is given to texture, structure and those impurities that may cause rock failure. Points to be guarded against in selection of aggregate are pointed out.—(From synopsis by Am. Soc. Testing Materials.)

The water absorption and penetrability of brick. J. W. McBURNEY. *Proc. Am. Soc. Testing Materials*, 1929. V. 29, Part 2, p. 711-730.—This paper records the values obtained from the determination of water absorption and penetrability by a number of methods on a variety of makes of brick representing a considerable range in properties and methods of manufacture. The value of water absorption as a measure of resistance to frost action is discussed. The mechanism of water transmission through masonry is discussed from the standpoint of absorption properties of the units. On the basis of the data recorded and field observations it is considered that the water absorption or apparent porosity of brick is not a measure of resistance to climatic action in so far as differences in raw materials and methods of manufacture are involved. However, for the same clay or shale and for the same method of forming, the difference between satisfactory and unsatisfactory weathering performance may usually be distinguished by difference in water absorption. It is believed that while Schurecht's ratio (water absorption, 48-hour cold immersion to water absorption, 5-hour boiling) is a much better measure of disintegration through freezing and thawing than is water absorption by any one method, it is not sufficiently reliable to furnish a basis for classification.—(From synopsis by Am. Soc. Testing Materials.)

Influence of quality of coarse aggregate on strength of concrete. F. C. LANG. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 811-815.—The tests and characteristics by which the quality of coarse aggregate is ordinarily judged are enumerated. The effect of each of these properties on the compressive, flexural and tensile strength of concrete so far as could be found in existing published research data is shown. This information is taken from investigations which were primarily made for other purposes. No comprehensive published research data pertaining to this particular subject was found. Conclusions drawn from the data available are: 1. Coarse aggregate deficient in structural strength as shown by the abrasion test

has more effect on the flexural and tensile strength of concrete than on the compressive strength. 2. The absorption in itself has no effect on the strength of the concrete when the effective water-cement ratio is correctly used. 3. There are not enough data to warrant any conclusions as to the effect of shape and surface texture.—(From synopsis by Am. Soc. Testing Materials.)

Modulus of elasticity and Poisson's ratio for concrete, and the influence of age and other factors upon these values. RAYMOND E. DAVIS and G. E. TROXELL. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 678-701.—The paper describes long-time laboratory tests made on plain concrete cylinders to determine the axial and lateral deformations produced by axial compressive stresses. Following the customary practice, the ratio of unit compressive stress to the corresponding total unit axial deformation is designated as the secant modulus of elasticity even though the deformations are not instantaneous in character and are not altogether elastic. The relation between lateral and axial strains accompanying compressive stresses in given by values of Poisson's ratio. The tests were made to establish the manner in which the modulus of elasticity and Poisson's ratio are influenced by (1) richness of mix, (2) age of concrete, and (3) repeated loads. Supplementary tests were made to determine the influence upon the modulus of elasticity of (4) moisture content of the concrete, and (5) length of period of sustained load. The paper presents the results of these tests and discusses the findings. Among other things, it is shown that the stress-strain ratio or modulus of elasticity is less for a concrete when dry than when wet, under a sustained load is greater immediately upon application of load and continues to decrease for a long period of time, is not necessarily highest for the richest mix and strongest concrete, and increases quite rapidly with the age of the concrete for the first year but after three years the increase with time is small. Poisson's ratio is greater for low stresses than for higher ones, it increases somewhat with age up to about 1½ years, after which it remains nearly constant, and does not seem to be influenced by strength of concrete or by richness of mix.—(From synopsis by Am. Soc. Testing Materials.)

The tensile autogenous healing of portland-cement mixture. HERBERT J. GILKEY. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p. 593-607.—In a former paper (cf. H. J. Gilkey, "The Autogenous Healing of Concretes and Mortars," *Proc. Am. Soc. Testing Materials*, 1926, V. 26, Part 2, p. 470) there were recorded considerable data upon compressive autogenous healing of portland-cement mortars and concretes. It was evident that, if compressive healing occurred, there must be tensile healing, since tensile, compressive and shearing resistance are, in general, but varied manifestations of the same thing. While a preliminary test referred to in the earlier paper failed to produce evidence of it, subsequent experiments have shown that tensile autogenous healing is a reality for mixtures of portland cement and water, with or without aggregate of usual accepted kinds. Healing occurs under the same conditions as initial moist curing and is essentially but a continuation or a resumption of the normal curing processes. It is evidently not in the least due to redeposition of soluble salts in cracks, as has been sometimes supposed. Standard tensile briquets of widely varying water-cement and cement-sand ratios (from 0.20 to 1.20, and from neat cement to 1:5, respectively) have been completely severed as many as six times at initial test ages of from 3 days to 27 months. All mixtures and conditions have shown measurable healing. Individual recoveries range from zero to 221 lb. at 3.5 months for a neat cement specimen initially testing 248 lb. at 3 days. Severed fragments must be closely fitted, but the amount of pressure between the halves appears to be unimportant if the crack be kept closed. Healing will occur upon reimmersion after months of drying out. It occurs in either running or stagnant water. In general the extent of healing

is less after each successive retest, although there were many individual exceptions. The majority of the specimens gave no strength at the fifth or sixth breaking, although many carried a few pounds. Knowledge of tensile healing is of practical use in various connections and in addition it should contribute to a better understanding of concrete. The healing is not related to an interesting form of crack closure that often appears in sidewalks. This crack closure may be related to the surface disappearance of cracks in air-exposed concrete as noted by Abrams, Earley and Hollister.—(From synopsis by Am. Soc. Testing Materials.)

Transverse test for concrete. *Rock Products*, Dec. 7, 1929, V. 32, No. 25, p. 60.—Preliminary work investigating influence of certain variables in molding of concrete specimens carried out at U. S. Bureau of Public Roads and reported by L. W. Teller, employed the following methods: (1) Rodding specimens with a $\frac{5}{8}$ in. diameter round steel rod, using the standard method for molding compression specimens; (2) Identical with method 1 except that each layer of concrete was rodded 50 times; (3) Spading with a spade $4\frac{1}{2}$ in. wide and 6 in. long. The concrete was placed in two layers and each layer spaded 20 times; (4) Identical with method 3 except that each layer of concrete was spaded 50 times instead of 20 times; (5) tamping with a 2x2-in. square wooden tamper; (6) Tamping with a 4x4-in. wooden tamper. The conclusions drawn are: (1) That both the rodding and spading methods used are equally satisfactory so far as strength, uniformity and ease of fabrication are concerned; (2) that the additional manipulation of methods 2 and 4 over that of methods 1 and 3 results in little or no improvement in the specimens obtained; (3) that the tamping methods are not as satisfactory as rodding or spading methods.—J. J. LANDY

Experiments about the influence of the addition of water on the apparent density and the compressive strength of concrete with pumice as aggregate. W. BIERHALTER. *Zement* (Germany), Dec. 1929, V. 18, No. 51, p. 1466-71.—Pumice of a particle size from 0-20 mm. was used as an aggregate for concrete. The determination of the moisture was very important for the requirement of water for normal consistency. The pumice contained from 5 to 20 per cent moisture. The absolute porosity p_a was calculated with the formula: $p_a = 100 \times \frac{D - r}{D}$; (D = 2.36 = spec. gravity, r = .55 = apparent density). p_a was 76.4 per cent per volume. The apparent porosity was calculated with the formula: $p_r = w_g \times r$; (w_g = 53.0 per cent = water absorption in per cent of the dry weight). p_r was 29.2 per cent per volume. This means that $\frac{3}{4}$ of the volume consists of voids, but only 3/10 of the total volume absorbs water. Testpieces were made with 1 part of cement, 2 parts of pumice from 0 to 5 mm. and 5 parts of pumice from 5 to 20 mm. particle size with a water addition from 25 to 35 per cent. The correct water addition is 30 to 32 per cent which gives the highest apparent density. The differences between the apparent densities decrease with the age of the testpieces. The water addition affects the compressive strength very much, $32\frac{1}{2}$ per cent give the highest strength. The hardening process proceeds very regularly.—A. E. BEITLICH

The effect of zinc and zinc oxide on the initial set of cements with a slag base. (See MATERIALS.)

Influence of the different kinds of sand on the mechanical properties of cement. (See MATERIALS.)

"Si-Stoff" and trass as aggregates for cement and lime. (See MATERIALS.)

Concrete construction during frost periods. (See FIELD CONSTRUCTION.)

Plastic flow in concrete arches. (See ENGINEERING DESIGN)

A compression test for portland cement. (See MATERIALS.)

Fine aggregate in mortar and plaster. (See MATERIALS.)

ENGINEERING DESIGN

Dams

Calderwood high arch dam designed with deep cushion pool to receive overflow. *Eng. News-Record*, Dec. 19, 1929, V. 103, No. 25, p. 954-958.—The main dam is a thin-section arch 230 ft. from lowest foundation to top of piers. The top 45 ft. of arch rings have a uniform radius and a uniform 25-ft. radial thickness. Below this level the radius decreases while crown thickness increases. Likewise radial thickness of arch ring varies from a minimum at crown to a maximum at the abutments. The upper portion of the dam supports 24 Stoney type gates 25 ft. long and 20 ft. high which control pond level and provide spillway area for 200,000 sec. ft. Gates are located at the downstream edge of the crest to add stability to the structure and also to deflect falling water as far as possible from the base of the dam. For construction purposes the dam is divided into radial blocks. Joints between blocks are closed by metal strips and will be grouted in. Arch deflection tests are to be made of the entire dam under various loads as the reservoir fills. Hooks are being placed and automatic equipment installed to measure deflections and to obtain water and air temperatures. Strain-gage readings are to be made on 40 rosettes buried in the concrete on downstream side, giving readings in vertical, horizontal, and two diagonal positions. Ten automatic electric thermocouples are being buried in the concrete at various depths to obtain data on temperatures of concrete setting. Stresses were computed in accordance with theory of elastic work, neglecting the elastic work due to shear, by a rapid method of design developed by the company's engineers.

To protect downstream toe of main dam, a reinforced-concrete armor of 3-ft. minimum thickness and concrete bucket deflectors were employed as well as a cushion pool, formed by means of a 40-ft. dam about 400 ft. down stream from the main dam. It is not curved in plan, but broken into three tangents, with a total crest length of 390 ft. The cushion pool will have a minimum depth of 35 ft. and a maximum of 65 ft. during full flood. A 240-ft. pressure tunnel has an intake structure of three vertical 15 x 24-foot gates. The first 220 ft. of tunnel is circular with a radius of 13 ft. 3 inches. The remainder has a 26 x 24 ft. section. The entire tunnel has a 12-in. minimum concrete lining. There are three penstocks at the lower end of the tunnel each carrying a steel pipe 16 ft. in diameter. At the head of the penstocks, a differential surge chamber, blasted out of solid rock, is unique feature. Surge shaft from tunnel to surge chamber is 18 ft. in diameter, has a 51° slope upward, and is of concrete. Surge chamber is 124 ft. long, 30 ft. wide, and from 42 to 55 ft. high. Three reaction type turbines of 56,000 H. P. capacity each are being installed in the power house on river bank.—N. H. ROY

Design and construction of Big Dalton multiple arch dam. E. C. EATON. *Eng. News-Record*, Dec. 26, 1929. V. 103, No. 26, p. 994-997.—Dam on the Big Dalton wash, about 30 miles east of Los Angeles, is of multiple-arch type with six arches supported by double wall buttresses, the highest one being 145 ft. above stream bed and 180 ft. above lowest foundation. The slope of the arch crown is 9 horizontal to 10 vertical and thickness of the highest arch varies from 5 ft. at base to 2 at top. Buttresses are 60 ft. center to center, and the highest buttress has walls 4 ft. 6 in. thick at ground level separated by a 5 ft. 6 in. space. This section gradually changes to a 2 ft. wall-thickness and a 10-ft. space near the top. Below ground level buttresses are solid with spread footings. Vertical stiffener walls, provided between the two buttress walls, are 1 ft. 6 in. thick spaced at 30 ft. intervals, and a concrete slab 2 ft. thick closes the downstream end of the buttresses and provides additional stiffness. Each buttress is considered to act as a series of H-section columns with a column ratio for the highest buttress of about 14 to 165 or approximately 1 to 12. No horizontal struts are deemed necessary. The two main limitations in the design were a maximum foundation pressure of 150 lb. per sq. in. and a minimum

concrete thickness of 24 in. Arches were designed by two current methods: Maximum compressive stresses in the arch due to combined water pressure and rib shortening are approximately 570 lb. per sq. in. at the intrados of the lowest arch. Load conditions as indicated by the design do not produce tension in the arches.

Two contractions joints were employed in each buttress parallel to the direction of principal stress. These planes of weakness are to localize possible cracks due to shrinkage or other tension stresses. Reinforcing was unusually heavy, amounting to 44.57 lb. per cu. yd. of concrete in the arches and 29.34 lb. per cu. yd. in the buttresses. Gravity abutments 5 ft. wide at the crest and with slopes corresponding to buttress slopes were substituted for arch rings because of heavy overburden and poor material encountered on one side. Steel centering was used to support the intrados arch forms. Only one hoisting tower was necessary. Strain-gage measurements were made on buttress reinforcement and over each diagonal contraction joint. Gage lines were installed after the joint had opened about 0.001 in. Subsequent measurements showed that the joint opened in cold weather and closed in warm; the largest movement was about 0.001 in. in addition to the original opening. This movement, had the contraction joint been omitted, would have caused tensile stresses of about 100 lb. per sq. in. Plaster of paris painting of joints showed fine hair cracks. No other cracks have appeared in the dam to date although temperature changes of from 30 to 100° have occurred. The contraction joint has so far fulfilled the design requirements. Design and construction were under the general supervision of the author. —N. H. ROY.

Bridges

Plastic flow in concrete arches. LORENZ G. STRAUB. *Proc. Am. Soc. C. E.*, Jan., 1930, V. 56, No. 1, Papers and Discussions, p. 49.—Mr. Straub calls attention to the fact that such a material as concrete violates the usual elasticity assumptions in two important respects: 1. There is hysteresis or flow under load. 2. The ratio of stress to elastic strain is not constant. In the analysis of indeterminate structures these violations of Hooke's Law have usually been ignored. Efforts to allow for them have been crude. The investigations show that precise measurements of strain will often lead to wrong conclusions if these factors are not correctly considered. In a highly mathematical treatment, the writer has evolved methods of analysis that are termed "the theory of plasticity" and "the proposed theory of elasticity." They are developed in connection with arches but are of general application. From the writer's studies the following conclusions are drawn: (1) It is incorrect to assume that in general plastic flow in statically indeterminate concrete structures causes a redistribution of bending moments tending to relieve the overstressed parts of the structure. (2) External disturbances of an indeterminate structure such as settlement of a support, temperature changes or shrinkage, will produce some redistribution but will not fully restore the original state of equilibrium. Without external disturbances plastic flow will progressively alter deformations but will have little effect on moment distribution. (3) Barring possible disturbances from reinforcing steel, there are advantages in removing the centering at an early age. The total rib shortening will be greater but the resulting stresses will be less. (4) In the writer's studies of data, plastic flow caused the line of thrust to approach the center axis. The reverse appears to be possible under suitable conditions. (5) For purposes of analysis of rib shortening stresses, it is important to distinguish between shrinkage, temperature, and elastic deformation on the one hand and plastic deformation on the other. (6) The violation of Hooke's Law does not greatly affect the moment distribution but for rib-shortening and temperature changes the usual methods may be greatly on the side of safety. (7) For small structures, the usual methods of analysis are probably satisfactory, but the new theories are needed in the case of important statically indeterminate structures.—H. J. GILKEY

Bridge at Bayview Heights—Toronto. H. A. BABCOCK. *Can. Eng.*, Dec. 31, 1929. V. 57, No. 27, p. 845.—Bayview Heights bridge over Don River, Toronto, consists of a 335 ft. earth fill between concrete retaining walls, a section of reinforced concrete slab, beam and girder design 125 ft. long, and a 735 ft. section made up of 5 spans of steel truss construction carrying a concrete deck and supported by concrete towers resting upon Raymond concrete piles. Concrete deck consists of roadway 24 ft. wide surfaced with asphaltic concrete with 6 ft. sidewalks on each side. Concrete towers or piers each consist of 4 rectangular reinforced concrete columns supported at three levels with deeply haunched stay girders.—G. M. WILLIAMS

Railway bridge has novel center pier. *Engineering and Contracting*, Dec. 1929, V. 12, No. 12. p. 522.—A bridge of ingenious design is situated on the new Nice-Conti line of the Paris, Lyons & Mediterranean Ry., at the crossing of the steep, narrow gorge of the Brevera River. The span is about 300 ft. Since the crossing is a skew crossing, it was found possible to construct a concrete arch, spanning between the walls of the gorge, at right angles to the centerline of the bridge and at the center of the 300-ft. span. Arch is 49 ft. high and has an 82-ft. span. A bridge seat at the crown serves as the center support for two steel lattice girders 147 ft. 8 in. long.—N. H. ROY

Bridge at Mill Hill. *Con. Constr. Engr.* (England), Dec. 1929, V. 24, No. 12, p. 738-740.—Two sets of precast reinforced concrete main girders with precast reinforced deck slabs were used for spans over double railway tracks in the five-span structure on Watford by-pass. Girders were designed as freely supported under their own weight and that of slabs. Reinforcing bars in precast members left projecting from girders lap with bars in remaining girder spans cast in place, enabling design on the basis of continuous girders for dead load on the remaining spans and entire structure as continuous for live load conditions. Main girders with 70 ft. span are 6 ft. on centers and 2 ft. 2½ in. below deck slab, with no secondary beams. Reinforced concrete deck slab is covered with plain concrete varying from 12 in. at crown of 30 ft. roadway to 6 in. at edge. Abutments, wing walls, and piers are mass concrete with foundations 10 ft. below rail level. Precast girders were handled on small undercarriages on narrow gage tracks and lifted by locomotive crane to concrete bearing pads cast on top of piers.—D. F. JENNINGS

Pier construction for the Mid-Hudson bridge at Poughkeepsie, New York. (See FIELD CONSTRUCTION.)

Miscellaneous Structures

Basement walls and floors. *Concrete*, Dec., 1929, V. 35, No. 6, p. 43-48. This article is a brief and practical discussion of the design and construction of concrete footings, floors and retaining walls. Discussion includes computation of footing loads, dimension of footings, forms for monolithic foundations, correct concrete mixes, watertight concrete walls, floor finishing, and placing and curing of concrete.—D. E. LARSON

Augmented steel. N. B. CARSON. *Structural Engineer* (England), Dec. 1929, V. 7, No. 12, p. 476-478.—The term "augmented steel" has been applied to a type of concrete-encased structural steel in which the concrete as well as the steel has been used in determining the strength of the member. A recent building in Glasgow employed 50 tons of steel including structural shapes and reinforcement when so figured, in place of 73 tons which would have been required if the concrete had been assumed merely as so much additional dead load to be carried by steel framework. A single test T-beam, crudely loaded, carried a load of three times the calculated safe load without failing or showing a permanent set.—V. P. JENSEN

Subsidence under tidal pressures. O. ARUP. *Structural Engineer* (England), Dec., 1929, V. 7, No. 12, p. 480-482.—Failure of a retaining wall reported in the September issue as due

to "subsidence under tidal pressures" was simply an instance of improper design. Reinforced concrete angle retaining wall supported by two rows of vertical or nearly vertical piles can be successfully employed only when there is sufficient earth in front of the wall to prevent the wall from gliding forward. One way to ensure the stability of a wall where there is insufficient passive earth pressure in front is to anchor it back by aid of ties connected to anchor blocks or continuous anchor beam at the back. Another method is to drive all the piles in the direction of resultant of vertical and horizontal pressures due to the weight of the wall and the pressures acting upon it. Practically this means driving the various rows of piles at different angles. Another method is to raise the level of the horizontal slab and construct the front wall below the slab as a sheet wall.—V. P. JENSEN

Welland ship canal. *Can. Eng.*, Dec. 17, 1929. V. 57, No. 25, p. 805.—Welland ship canal 25 miles long between Port Welland and Port Colborne, Ont., on shore of Lake Erie, will be completed in 1930. Includes concrete locks each having lift of 46½ ft., built for a 30 ft. draft. Construction involves use of 3,516,000 cu. yd. of concrete of all classes. No details covering concrete materials or practice.—G. M. WILLIAMS

Large gravel pits at Hatton. (See FIELD CONSTRUCTION)

ARCHITECTURAL DESIGN

The New Architectural Medium—Concrete. FREDERICK A. HANSON. *Architect and Engineer*, Jan., 1930, V. 100, No. 1, p. 89-98.—"It is not wholly surprising that in California, the land of many unusual things, a significant architectural development should have taken place within recent years. . . . So it is that concrete—a powerful structural medium—has been adopted by Pacific Coast architects and transfigured to serve their purposes. A very remarkable transformation, this, but after all, a logical one. For there is beauty in honest strength. It remained only for architects to understand and develop the infinite possibilities of concrete as a medium for architectural expression. It is, indeed, a major event, the coming forth of concrete into a primary architectural position. It places at the command of the artist a material of very great adaptability and it presages much in the improvement of concrete itself. As a structural material it achieved an extensive use years ago, but, as with other materials, in a lowly utilitarian station. The dignity, nobility, and aspiration of the recent ecclesiastical uses of concrete are not be be found in its beginnings. Hitherto, the fact that concrete was either to be concealed . . . or was left indifferently exposed to view on alley or property line, led to carelessness in construction methods, to slipshod form work and sloppy mixtures that many times were appalling in their ugliness. Who has not seen the story of carelessness written in concrete walls? . . . Again on the other hand, who has not seen walls which, though devoid of relief, . . . in the sound, unvarying quality of their concrete and in the impressed record of the forms, testify to work done with all the skill and with all the spirit which good workmen have ever shown? It is with concrete of such a character, the physical nature of which is not different in one part of the work from its nature somewhere else in the structure, that the successful new work has been done. Plastic, mobile, sensitive to the most trifling variations of form and surface with which it comes in contact; yet in a few hours losing these properties forever, here is a material to engage the interest and fascinate the imagination. With textures and surfaces fresh with the natural variety of form boards, with random slight imperfections of plane and with line warm with the suggestion of the human hand and touch, with simple or elaborate ornamentation made integral with the structure. . . . and with an incomparable adaptability to mass effects, concrete has attained an architectural development and recognition warranted by its own inherent worth. It may be "fashioned to the heart's desire." Yet another impelling reason for its use is the satisfying honesty and sincerity of this construction. It is the structural frame itself which stands revealed, unconcealed and unashamed, and upon which

the architect exercises his ability of shaping mass, line, and ornament. It is the structural frame which meets the eye, not that which appears to be the structural frame. In what indefinite and subtle manner this distinction between the actual and the apparent is expressed is not always easy to explain, but the distinction, none the less, exists and declares itself. It can be for no other reason than that, from its beginnings, architectural criticism has ever emphasized its virtues of honesty and sincerity in construction. . . . It is hazardous to prophesy, yet it would appear that this architectural use of concrete will soon find a general and wide acceptance. That this should be is but the logical consequence of proven and demonstrated success." (Abstractor's Note: The article is illustrated by the Wiltshire Professional Bldg., Richfield Oil Bldg., First Baptist Church, St. John's Church, Church of the Precious Blood, and the Ebell Club, all in Los Angeles.)—REXFORD NEWCOMB

New viewpoints for the prevention of fire catastrophes by means of constructive and fire preventing installations. (See FIELD CONSTRUCTION.)

FIELD CONSTRUCTION

Bridges

Pier construction for the Mid-Hudson bridge at Poughkeepsie, New York. JAMES W. ROLLINS. *Journal of the Boston Society of Civil Engineers*, Oct., 1929, V. 16, No. 8, p. 423 to 448.—The plan for the suspension bridge at Poughkeepsie, for the Dept. of Public Works, State of New York provided for a river span of 1500 ft., two shore spans of 750 ft. and a clearance above the river of 135 ft. Construction of two river piers 60 ft. wide, 130 ft. long and extending in one case to 135 ft. and the other to 115 ft. below the water level. The open caisson method was to be followed, using special caissons designed by the engineers.

The caissons consisted of an outside concrete shell with reinforced concrete cross and longitudinal walls making 25 pockets of full size from top to bottom. The bottom 21 ft. of the cassion acting as a cutting edge, was provided with a ½-in. steel shell and structural steel bracing. These pockets were provided with false bottoms of timber to be removed later.

The bottom sections of the cassions were built at Staten Island, towed to the side and there the concrete for the walls was placed in 16 ft. lifts as the cassion sank. The forms were of wood and were made in 16 ft. high by 15 ft. long sections on shore, taken to the cassion and bolted to the structural steel framework. Concrete was placed from a floating plant mounted on a car float 40 ft. wide and 120 ft. long. Aggregates were received by scows and unloaded to a storage bin on the car float using a clam shell. Cement was received in covered barges from a cement mill 40 miles up river and unloaded by belt conveyers delivering to two 1-yd. mixers. Belt conveyers were used at first to place the concrete in the walls but the movement of the barges caused difficulty in their operation and so chuting was adopted. When the East pier came to rest on the river bottom considerable difficulty was encountered in trying to remove the false bottoms and to get the caisson to sink further. After the cutting edge was 10 ft. into the river bottom, the bottom broke and the caisson sank suddenly 30 ft. and tilted to an angle of 43° to the east leaving only one side above water. Three months were spent trying to right the caisson without success. The contractors were then allowed to try a plan they had evolved which consisted of the application of a turning couple and excavation under the high side. The righting couple was produced by a horizontal pull, by counterweights, by the addition of concrete on the high side and by lifting on the low side. This plan was successful and the caisson was finally righted about a year after it first tilted. Trouble was encountered in sinking the West caisson as soon as the first section of the false bottom was removed. The caisson took a list to the southwest and while attempting to correct this by removing the false bottoms in other pockets, the caisson moved 7 ft. to the west. This condition was corrected by a surcharge of rock which gradually

forced the caisson back into position as the regular procedure of sinking continued.

After the caissons were down to grade and the foundation concrete placed, the pockets were filled under water in a continuous pour of from 600 to 800 cu. yd. per 24 hours using a 2-cu. yd. submarine bucket. At the completion of this 86-ft. pour, work was stopped until the next spring.

When the new work was started, which was to be done in the dry using a removable cofferdam, it was found that there was from 20 to 30 ft. of mud, silt and laitance on the old pour which was so hard jack hammers had to be used in its removal. Work was completed to grade without further difficulty. In all 72,300 cu. yds. of concrete and 700 tons of reinforcing steel were placed.—MILES N. CLAIR

Bridge at Mill Hill. (See ENGINEERING DESIGN.)

Dams

Island Falls hydro power development. F. S. SMALL. *Can. Eng.*, Dec. 3, 1929, V. 57, No. 23, p. 765.—Hydro power development, initial capacity 44,500 H. P. ultimate 80,000 H. P. now under construction at Island Falls on Churchill River about 60 miles from railroad is most northerly power development in Canada. Main dam is of concrete gravity type with four gate controlled under sluices 12 ft. wide and 24 ft. high, and 16 stop log controlled spillway openings each 15 ft. 6 in. wide. Also a concrete gravity spillway dam with 46 stop log controlled openings. Concrete in power house superstructure main dam and spillway dam total 80,000 cu. yd. Walls of power house made of concrete blocks molded at site. Materials were freighted in by water during summer and over land in winter.—G. M. WILLIAMS

Design and construction of Big Dalton multiple arch dam. (See ENGINEERING DESIGN.)

Roads

Road construction without detouring. W. A. VAN DUZER. *Eng. News-Record*, Dec. 5, 1929, V. 103, No. 23, p. 884-885.—Preliminary study of conditions governing the improvement of Lincoln Highway section showed one of most important factors affecting delay to traveling public at given operation was length of roadbed over which single file traffic had to be maintained in avoiding long detours in the absence of parallel road. Opening of Pennsylvania pavements to traffic is based upon modulus of rupture developed by test beams molded simultaneously with pavement. Minimum set for this project was 550 lb. secured in 10 to 14 days with standard paving mix of 1:2:3½ standard portland cement concrete. Conditions required high early strength concrete for equivalent to 10 ft. strip 15 miles long. High early strength portland cement was used on first strip and two-way traffic maintained at all times except in immediate vicinity of mixer, concrete more than 24 hours old being used for one traffic lane and subgrade for other. Sections of high early strength pavement alternated with standard concrete in second half-width of highway. Test beams poured daily were tested in 24 hours for high early strength and at periods from 10 to 14 days for standard concrete. All 24-hour concrete was cured by use of 2 thicknesses of saturated burlap placed as soon as concrete had hardened sufficiently and concrete was kept wet for 24 hours after which curing ceased and pavement was opened to traffic. A 2 per cent solution of calcium chloride was used in mix of normal concrete and pavement kept wet for 24 hours assisted by burlap covering, after which it was allowed to cure without further wetting until tests indicated sufficient strength to permit its use by traffic. Conclusion based upon studies during construction was that method would have been economical even for lower traffic volumes. Added expense of carrying vehicles over right-of-way would have been justified for average daily traffic volumes as low as 800 vehicles, one third of that shown by traffic count on this road. Work was carried out under directions of H. E. Kloss. J. I. Dick was contractor engaged in planning details and execution of work.—D. F. JENNINGS

Miscellanous Structures

Heavy construction on new line of L. & N. R. R. *Eng. News-Record*, Dec. 12, 1929, V. 103, No. 24, p. 916-919.—Construction of tunnels on 13½ mile branch through Cumberland Mountains in Kentucky involved concrete linings with materials obtained 244 miles from site of work and dumped from main line above tunnel portal. One-yard mixer plant on top of approach cut at Hagan's tunnel was charged from dump cars on overhead trestle, cars being loaded from stock piles by stiff leg derrick with clamshell bucket. Concrete was placed by hand from standard flat car carrying scaffold and platform at two levels. Concrete was conveyed from mixer to cars in hopper bucket hanging from carriage and operated on 45° incline by air hoist. Scaffold car was handled by standard locomotive and forms completely rebuilt for every pour. Pneumatic concrete-placing machine is to be used in completing lining. All plant and equipment was dragged over hills by tractor to working sites. Waterways and existing mine structures were crossed by permanent structures involving concrete bridges. Three 70-ft. arches required a 3-mile temporary standard gage railroad from spur line. Several bridges on connection between lines near Hagans were constructed of concrete mixed by plant at south Hagans portal and transported in hopper mounted on flat car.—D. F. JENNINGS

New underground railway offices, St. James Park. Discussion, MAJOR F. M. DU-PLAT-TAYLOR AND OTHERS. *Structural Engineer*, (England), Dec., 1929, V. 7, No. 12, p. 469-473.—In using tubular piles in which the tube was withdrawn after the concrete had been cast, previous experience had shown that when a strata of peat was penetrated a distinct "waist" formed in which the concrete was compressed from a 15-in. to a 6-in. diameter. Other difficulties were displacement of the reinforcement and segregation due to placing concrete from a great height. In the Vibor piles, which were used for the Underground Railway Office, reinforcement was kept central by barbs spreading out from the hoops to the inside of the tube, the barbs having been spaced 10 in. on center on each main bar. When very long Vibro piles were necessary they were made up in two parts. Concrete placed by the Vibro system did not show indications of segregation in piles up to 30 ft. in length, which were dug out. A "waist" is unlikely to form in strata other than peat, which has the consistency of India rubber.—V. P. JENSEN

Storm protection for lakefront building in Chicago. *Eng. News-Record*, Dec. 12, 1929, V. 103, No. 24, p. 920-921.—Concrete bulkhead of wave—deflecting type built along lakefront property line encloses old structure of wood piles and sheeting capped by low concrete wall. Line of 20-ft. interlocking steel sheet piling driven through sand into clay and left projecting 18 in. above water level formed the face of a 6-ft. trench excavated down to clay. Tamped and puddled sand and broken rock were filled to 2-ft. below water level for foundation for reinforced concrete wall. Reinforcing rods were anchored to steel sheeting. Sloping surface of front of wall curves outward. Top of old curb wall was partly broken away to provide rough surface for bonding with new concrete.—D. F. JENNINGS.

Transit system of mixing concrete—Toronto, *Can. Eng.*, Dec. 31, 1929. V. 57, No. 23, p. 775.—Toronto Ready Mix Concrete Co. Ltd. has established a bunkering plant for supplying materials to the Paris transit mixer. Aggregates stored in elevated bins of 400 cu. yd. capacity and proportioned by Johnson weighing batchers. Cement is conveyed by electrically operated screw conveyor and proportioned by same type of weighing batcher. Filtered water with provisions for heating is supplied through measuring tanks. For winter operation aggregates are heated by grillages of 2-in. perforated pipe and water tanks on truck are heated by exhaust gas from engine. In operation aggregates and cement are fed to mixer drums while rotating and truck water tank is supplied with necessary quantity. Five minutes before arriving at destination

truck operator starts drum revolving and opens water valve. Preliminary study of local materials together with complete control of batch quantities and frequent tests of finished output as delivered result in production of uniform quality concrete of any desired grade.—G. M. WILLIAMS

Large gravel pits at Hatton. JOHN E. ADAMS. *Conc. Constr. Eng.* (England), Dec. 1929, V. 24, No. 12, p. 701-708.—Structures completed for operation of large gravel deposit provide crusher installation separate from main plant with separate storage bins and bunkers. Reinforced concrete ground storage bins have 15 ft. back wall and 6 ft. front wall connected by cross walls. Other concrete work includes settling tanks and conveyor pit. Water caused main difficulty in foundation and pit work. Ransome sheet piles were driven around site of pit and timbered trenches along side piling led to sump from which water was pumped. Mass concrete 3 in. thick was deposited over entire bottom of pit and sump pump operated all night until concrete had hardened. Steel reinforcing was placed at night and reinforced concrete floor placed next day. As soon as first 3 ft. 6 in. lift of side wall forms was erected, concrete was placed and water was allowed to remain inside pit to counteract buoyancy until pit could be finally held down by ballast on projecting toe around deepest part. Side walls of pit were designed as cantilevers continuous with bottom, and end wall as horizontal beam spanning between ends of side walls. Front walls of overflow bins were designed as cantilevers and back walls partly to span horizontally between cross walls and partly to act as cantilevers near the bottom. Forms for bins were made in 15 ft. lengths, 3 ft. 9 in. deep held in place by carefully aligned vertical timers. On completion of walls of settling tank, sump was filled by placing concrete through pipe. Rapid hardening portland cement was used throughout in proportions 1:2¼:4½. Reinforced concrete work was designed by the author.—D. F. JENNINGS

New viewpoints for the prevention of fire catastrophes by means of constructive and fire preventing installations. DR. NOLTE. *Zement* (Germany) Dec. 1929, V. 18, No. 49, p. 1421-3.—Outlines are given of methods for preventing fire catastrophes. Iron or steel structures must be protected sufficiently by masonry. Ceilings should be built of fire resisting material and not of wood.—A. E. BEITLICH

Modern concrete distributing arrangements. HOLZ. *Zement* (Germany) Dec. 1929, V. 18, No. 49, p. 1424-5.—A modern cable crane for the construction of a dam is described. The crane was built under unfavorable surface conditions and handled 2000 cu. meters of concrete daily.—A. E. BEITLICH

Something about the practice of solid constructions. LUZ DAVID. *Zement* (Germany) Dec. 1929, V. 18, No. 48, p. 1396-1401.—Examples are given showing how small mistakes in the construction of buildings can be of great importance to the durability of the material.—A. E. Beitlich.

Concrete construction during frost periods. A. DAHLGREN. *Zement* (Germany) Dec. 1929, V. 18, No. 50, p. 1446-7.—A traveling crane base was built during a strong frost. The influence of the addition of NaCl to the water on the compressive strength and the setting time was studied. An addition of 5 to 10 per cent gave the best results without any considerable effect on the compressive strength. Additions of 10 to 30 per cent decreased the strength very much. The temperature during the construction was from 0° to —17.7° C.—A. E. BEITLICH

SHOP MANUFACTURE

Manufacture of art objects of colored cement. GEORGE RICE. *Rock Products*, Dec. 21, 1929, V. 32, No. 26, p. 113.—Part III of series on manufacture of concrete tile inserts for metal backgrounds deals with tile table tops made of mixture of 1 part portland cement to 3 parts sand. Designs are first cut out of cardboard and used for preparing the molds of casting plaster. —J. J. LANDY

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